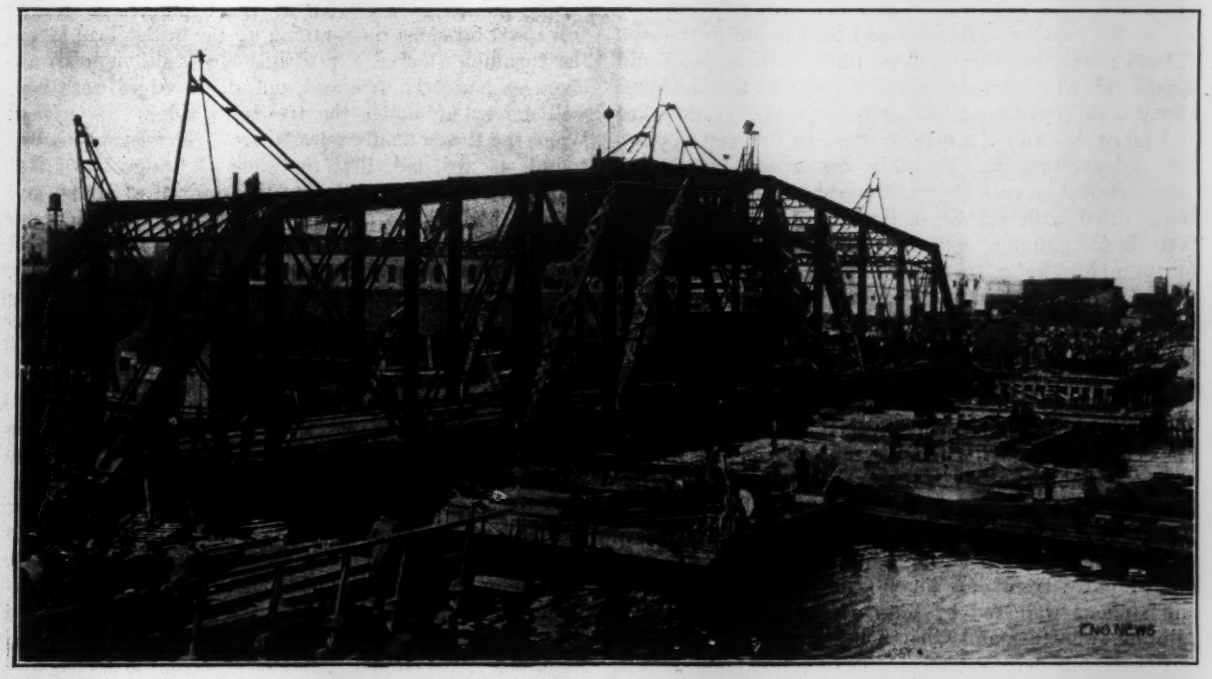
Engineering News

VOLUME 72 NOVEMBER 26, 1914 NUMBER 22

Shifting the Milwaukee River Drawbridge; C. & N. W. Ry.

The shifting of the 500-ton, 242-ft. double-track swing bridge carrying the main line of the Chicago & Northwestern Ry. over the Milwaukee River, at Milwaukee, Wis., was accomplished successfully on Oct. 25. The bridge was lifted off the old center pier and abutments by means of barges or scows, moved downstream about 60 ft. and placed on a temporary pier about 1 ft. above the former elevation. The time consumed was four hours, and the bridge was out of service for trains for five hours.



SHIFTING THE MILWAUKEE RIVER DRAWBRIDGE OF THE CHICAGO & NORTHWESTERN RY. AT MILWAUKEE, WIS. (OCT. 25, 1914)

(The two scows are floating the bridge down to the temporary center pier at the right of the bridge. The crowds are standing on the new temporary approaches. A group of men with one of the hauling lines is just beyond the near end post. Note the diagonals of the panels next to the center, with timber blocking to form compression members.)

The old bridge is too light for modern wheel loads and train loads, and the foundation must be remodeled for the new bridge. The line has been diverted temporarily, therefore, to leave the site clear for reconstruction, and the bridge was shifted to this temporary location. The river is about 350 ft. wide between dock lines, and the bridge crosses on a skew of about 10°, giving two 85-ft. openings with about 20 ft. of water. It had pile approaches of about 100 ft. at the north and 30 ft. at the south end. The skew and the existence of the protection pier made it necessary to arrange the scows carefully to insure clearance when moving the bridge, particularly as they were placed normal to the bridge in order to avoid undue stresses in them.

Two scows were used; one of steel 100x35 ft., and 8 ft. deep; the other of wood 112x33 ft., and 9 ft. deep. Each had four transverse and four longitudinal trusses (the latter in addition to the heavy side frames), while the steel scow had also a longitudinal and transverse watertight bulkhead. While the 500-ton load could be carried safely as a distributed load upon either one of the scows employed, each having a carrying capacity of about 100 tons per foot of depth, such scows are not designed for such heavy concentrated loads in the middle as would be caused by lifting a bridge. Therefore, all displacements and stresses in both scows were calculated carefully. This showed the necessity of reinforcing the splices in the bottom chords of trusses in the wooden scow, which was done by placing long cheek planks on both sides of the chord timbers at splices and bolting them up well.

To secure greater height of blocking for better distribution of loads under these conditions, the blocking was placed against the stringers instead of the floor-beams; this afforded 2 ft. greater height.

As the turntable (with wheels, center casting, rack and tread) was bolted up to lift with the bridge, and as the

scows had to be placed at a considerable distance from the center, on account of the skew of the protection pier, a reversal of stress took place in two pair of diagonal braces on each side of the center. These diagonals were eye-bars and there were no counters in the panels. It became necessary, therefore, to stiffen the eye-bar diagonals to enable them to take compression. This was done by wedging timbers between them and placing other timbers on the outside, all bolted together and laced with 2x8-in. plank, as shown. This bracing will remain in place for the second lift when the bridge is removed finally, after the new bridge has been completed and put in service.

The bridge is so near the mouth of the river that during gales on the lake waves as high as 2 ft. are produced at this location. It was therefore essential to make the movement when lake conditions were good, as with barges or scows filled with water to make the lift, any sudden extra load, thrown unequally on the scows, due to the surge in the river, might have serious results. Owing to a storm, a surge about 18 in. high existed at the bridge site the evening of Oct. 24, but the wind died down during the night, and conditions were favorable Sunday morning (Oct. 25). The work was decided on at 6:45 a.m., and by 8 a.m. the floating equipment had been towed to the site.

Both scows had their pumps, lines, pulleys, jacks and wedges, also as much blocking in place as the clearance allowed, the remainder having been put up at the yard and taken down by the marine crews in the way of practice and preparation. Two 6-in. centrifugal pumps had been installed on each scow at the end hatches, the pumps being fitted with rubber suction pipes so as to be adjustable for pumping water out for raising the scows or for pumping water in to sink them. Large steam siphons were placed in the end hatches to blow out any remaining water that could not be removed with the pumps, which were to be stopped when there remained only about a foot of water in the scows.

The hatches of both scows had been fitted with coamings about 18 in. high, to provide against water running over the decks and into the hatches, due to possible listing. The steel compartments in the steel scow and the partial longitudinal bulkheads placed temporarily for this purpose in the wooden scow further contributed to safety against listing, by preventing large amounts of water running rapidly from one side to the other. All pumps and siphons were fed by lines of armored steam hose, running from tugs and other floating plant, such as derrick boats, pile-drivers, etc., capable of giving ample steam, and moored both above and below the bridge before it was thrown out of service.

At 8:30, filling of the scows was commenced, to sink them enough to give ample capacity for lifting the bridge, also for the 1½-ft. extra raise on the new foundations, and to take care of a drop of about 6 in. in the river level, due to change of wind. The scows were moored on opposite sides of the river, one upstream of the bridge and the other downstream. Pumping was discontinued after 30 to 40 min., when the scows were filled to about their working limit, giving 16 to 18 in. of freeboard and 6½- to 7½-ft. draft. At 10 a.m. navigation was closed and the bridge was partially opened to allow the scows to be shifted endways into place under the bridge; the height of centrifugal pumps and engines prevented the passing of equipment under the bridge when closed.

With the scows finally lined up and well moored by numerous lines, and all clearances against protections, etc., carefully attended to, some further blocking could be placed, but no jacks or wedges could be fitted until the last train had passed in each direction. This was at 11:05 a.m., after which the bridge was turned over to the contractors for removal.

A partial strain was put on the scows by means of railway jacks against the bridge, and the placing of final blocking and wedges was completed. The wooden barge in the north channel was completely blocked and wedged at noon, this being the easier scow to block, as it was on a perfectly even keel, and also had a slightly more expert crew. The steel scow used in the south channel had a slight list toward the middle of the bridge when floated into position, due to the unequal weight of pumps, which were not placed on the center line of the scow, but at the diagonal corners, on account of the hatches. A listing effect is easily produced in a steel barge when well filled with water, making it a less stable craft. In this case, however, it was merely an inconvenience in the quiet conditions of river, requiring slower and more careful wedging against the bridge stringers.

The pumps were started at 12:30 p.m., and worked full speed for some time, taking up the bridge load before the turntable showed any lifting effect, although all anchorages had been removed and steel wedges were kept well driven up under the treads to aid in the release. When the treads finally released from the masonry, it was found, as expected, that the ends of bridge had lifted more than the middle, due to the reversal of stress, and largely, perhaps, to the play in the worn pins and pin holes. The pumps were stopped about 1:15 p.m., and siphons then blew out most of the remaining water. This was completed at 1:30 p.m., the bridge then being ready to move down the river. The turntable showed a lift of 27 in. from its original position, and the ends slightly more.

The wooden scow had 4 ft. of freeboard and an average of about 10 in. of water, which was hard to remove on account of the 12-in. bottom timbers partitioning the water off into pockets. The longitudinal trusses of the scow deflected about 3 in. at the middle. The steel scow had an average of about 34 in. freeboard and only ¾-in. deflection. It had about 4 in. of water in the bottom, this being difficult to remove on account of the watertight compartments, although temporary bypasses had been made in the steel bulkheads to equalize filling and pumping. The actual displacements of the scows conformed very closely to the advance calculations. The intention was to remove all water as nearly as practicable from both scows, preparatory to any moving of the bridge, in order to minimize the danger of listing.

No tugs or other craft were allowed to pull on the scows in shifting the bridge, these movements being made entirely by the use of numerous hand lines, which accurately controlled the movements. A competent man had been put in charge of each scow, with several marine hands at each line to follow the various directions carefully. All this was necessary, as the shifting of the bridge was not a simple downstream movement in a straight line, on account of the skew of bridge and lack of clearance for inner corners of scows along the bridge protection.

On account of the way the scows had to be placed, to take their loads, considerable maneuvering was necessary.

The old south abutment and the first span of its approach were removed to allow the bridge to be shifted south about 6 ft., after a swinging movement at the southeast corner, to clear the protection pier. The third shift was made downstream in the direction of the axis of the protection pier, until the north end of the bridge reached the center line of the new location. A slight northward movement was then necessary, the south end being allowed to continue downstream and pivot about the center of the north end, which was held stationary until the entire bridge brought up on the center line. The two ends of the bridge were then drawn up against stop timbers which centered it for alignment; wedges at the north end centered it endwise. The bridge was then held in this position ready for lowering.

The above operations took only 15 min., and the engineers then verified the location of the center of the turntable, etc., so that within 30 min. the scows were ordered filled. Pumping was started at 2 p.m.; the scows were filled in about 30 min., and wedges taken out at 3 p.m., the bridge then being in proper position on its abutments. The scows were moved out at 3:30 p.m. Track work and adjustment of ends of bridge continued, so that it was possible to let the 4 p.m. passenger train over the bridge in its new location without delay.

During the reconstruction of the foundations, the new bridge will be erected on falsework in the middle of the river and upstream from the site. This falsework will form an extension of the protection pier, and the bridge will stand parallel with the stream. It is expected that the foundation work will be completed so that early in January the new bridge will be placed in position. It will be raised by barges, moved upstream to allow of turning it crosswise of the river, and then floated down over the falsework to the new pier. It will be at the same elevation as the old bridge, with rail level 8.72 ft. above the water level or U. S. Government datum, but it will be erected on falsework 1.5 ft. higher. When this is in service, the old bridge will be removed again on barges to a distance of about 1000 ft. and there dismantled.

The old 500-ton bridge was built in 1890 and is operated by a gas engine. The new 800-ton bridge will be operated by electric motors. The piers in both cases are of stone and concrete on pile foundations, the old pier being 32 ft. square and the new pier 34 ft. diameter (octagonal). The Cleary-White Construction Co. has the contract for the substructure; the American Bridge Co. will furnish the machinery and structural material, and the Bernhisel Construction Co. will erect the superstructure, while Geo. P. Nichols & Bro. will install the power equipment. The moving of the old bridge was done by the Great Lakes Dredge & Dock Co. All these are Chicago firms. All the work was under the direction of W. C. Armstrong, Engineer of Bridges, Chicago & Northwestern Ry.

Telephone Service Rules of the Wisconsin Railroad Commission

Standards of quality of gas and electric service have been in force in Wisconsin since 1908 (see ENGINEERING NEWS, Oct. 8, 1908), but only recently has it been practical to make definite requirements of the telephone utilities, although the subject has been carefully studied all these years. The following rules were adopted Aug. 13, 1914, and are thought to be the first standards formulated—outside of the operating companies themselves. The rules are intended to fit large and small exchanges. They are on trial, and in cases of their resulting in hardship or increased cost they will be modified.

Line Construction—Rule 1. Equipment and lines shall be so constructed and maintained as to eliminate all cross-talk and noise, which unreasonable interferes with the transmission of messages for ordinary distances.

Rule 2. The number of subscribers on any one line shall not be greater than that consistent with adequate service.

(Under ordinary circumstances, rural lines should be limited to 10 or 12 subscribers, but in special cases a larger number may be justified. Service of a higher class should be rendered to subscribers who demand it and who are willing to pay the additional cost thereof.)

Rule 3. Each utility, furnishing service alone or jointly with other utilities in two or more cities, villages or other exchange points, shall provide at least one line for through traffic between such points, along which few if any subscribers' instruments are installed.

Rule 4. Each utility shall maintain in proper condition the lines, instruments and other equipment used on its system, and shall make such tests and inspections as are necessary.

Central Office—Rule 5. Each exchange shall have sufficient switchboard capacity and a sufficient operating force to handle the traffic at all times with reasonable facility. Traffic studies shall be made and recorded of such extent and frequency as to demonstrate to the commission that sufficient equipment is in use and that an adequate operating force is employed.

(Where operation is not continuous, provision should be made for handling emergency calls during such hours as the exchange is closed for regular service. Traffic studies should include the number of calls made each hour, the hour of heaviest traffic, the variation of these conditions with the day of the week or month or with seasonal conditions, and the distribution of calls among operators.)

Rule 6. Reasonable provision shall be made against the failure of lighting or power service, fires and storms, sudden increases in traffic, the illness of operators, or other emergencies which would seriously impair the service if not promptly met.

Rule 7. At exchanges serving 500 or more subscribers, 94% of the calls should be answered within 10 sec. or less. At all other regular exchanges, 90% of the calls should be answered within 10 sec. or less. At small exchanges operated in connection with other work, slower service may be adequate. Calls shall be carefully supervised and parties disconnected promptly after conversations are completed.

Rule 8. Suitable rules and instructions shall be adopted covering the phraseology and methods to be employed by operators in handling regular and special calls.

(Patrons should be required to call by number wherever practicable. It is advisable for operators to repeat the number. Telephone courtesy is extremely important in stimulating the growth of the business, and in eliminating dissatisfaction and complaints. Employees must not "listen in" on lines except when it is an operating necessity. Care must be taken to avoid diverting business from a subscriber to his competitors, or discriminating between subscribers in the attention given their calls.)

Subscribers—Rule 9. Directories in which 1500 or more subscribers are listed shall be revised at least semi-annually. All other directories shall be revised at least once each year. All directories shall be dated.

Rule 10. Directories shall contain such instructions and rules governing local and toll service and methods of payment as may be necessary to inform subscribers of their rights and obligations.

Rule 11. Reasonable efforts shall be made to eliminate interruptions and irregularities, and to correct them promptly when they occur. Records shall be kept of all complaints or irregularities in the service, showing the day and time at which the trouble is reported, the nature of the trouble, its duration and final disposition.

(Preservation of the ordinary slips filled out when the trouble is reported and the notes of the trouble man or wire chief with reference thereto will be sufficient. Subscribers should be given instructions as to how and to whom they should report difficulties. Employees should be encouraged to report all complaints, irregularities and criticisms. The coöperation of the public in promoting good service should be encouraged in the matter of tree trimming, in refraining from monopolizing party lines and in other matters not wholly within the control of the utility.)

Rule 12. The name and address of the official or employee designated to handle service matters and a copy of each new directory shall be filed with the commission. Upon request, a complete map of each telephone system shall be filed with the commission, and a similar map shall be kept at the principal office of each utility and revised from time to time as changes in the system are made.

Toll Service—No specific rules are prescribed, but the following suggestions are offered. Toll service should be properly routed so as to be most efficient and to secure justice to the telephone companies where more than one utility is involved. In general, each utility should test all toll circuits early each morning and after storms in order that trouble may be promptly eliminated. On joint lines, or when one utility uses the lines of another utility, trouble on circuits should be promptly reported to the utility responsible for the maintenance of the line. Accurate and convenient devices should be installed in order that toll charges may be just, and that the service may not be unnecessarily delayed on this account. Operators should cultivate not only a distinct articulation, but low tones and pleasing voice. A record of the condition of long-distance circuits entering each exchange should be kept for the convenience of the utilities in properly maintaining their lines, and for the Commission's information.

A Freight-Handling Equipment for Fertilizer, Central of Georgia Ry., Savannah, Ga.

By E. M. Rhett*

In this country in the past, the application of machinery to cheapen and expedite miscellaneous freight handling has not been as fully developed at seaports as at other points. This is not because our seaports, as links in the chain of transportation, are new, but simply because our attention has necessarily been occupied by other phases of the problem. The insistent demands of traffic are requiring a better solution. With present-day methods of transportation, a seaport must be regarded as primarily a joint terminal for railroad and steamship. We have been occupied heretofore more particularly with the development of the railroad part of the transportation problem; and it must be conceded that we have been conspicuously successful, since our open-road service and inland terminal handling are by far the cheapest in the world.

American engineers are now attacking the seaport terminal problem in a systematic manner. There has been quite a marked tendency to develop ports in self-contained terminal units; that is, each unit including all essential terminal elements, such as railway yards, docks, warehouses, etc., and each unit also being of sufficient size to give a good working whole. The terminal of the Central of Georgia Ry. at Savannah, is such a unit.

A study of this terminal was made about two years ago with respect to three classes of commodities: Fertilizer material imported from foreign countries; miscellaneous (or so called package) freight, handled by coastwise steamers to and from Boston, New York, Philadelphia and Baltimore; and export cotton. As a result of the study a general plan of development was prepared and certain improvements instituted. Fig. 1, a general layout of this terminal, embodies these plans so far as work which has been completed or is in progress is concerned. The improvement with respect to package-freight handling is under way; berths 1, 2 and 3 are completed and the building of slip No. 2 and adjacent facilities is in progress.

The improvements in the cotton-handling facilities are still in the initial stages, though there has been an interesting development in the application of motor trucks for handling cotton. The improvement as to imported fertilizer material is essentially completed and tested, except as to the final ship-unloading arrangements. This article is concerned only with this plant. Although not remarkable for size, the fertilizer plant is interesting as an exemplification of seaport conditions.

Characteristics of Fertilizer

The imported fertilizer material delivered at Savannah comes in two forms, in bulk and in bags. The bulk material consists mainly of various grades of kainite, a siliceous sand; it weighs approximately 85 lb. per cu.ft., highly impregnated with various salts of potassium. It is mined only in the vicinity of Stassfurt, Germany, and is brought down the Elbe in barges and shipped from the port of Hamburg. It is highly hydroscopic, absorbing moisture from the air in a surprisingly copious and persistent manner; when in this condition it is an excellent conductor of electricity. When handled carefully in loading it comes from the ship fairly dry, but during handling it gives off considerable quantities of dust which is almost pure salt. This, of course, settles and absorbs moisture when atmospheric conditions are favorable. Special precaution must therefore be observed in the protection of any appliance which may be used in handling it. Another physical characteristic which makes it difficult to handle is its tendency to cement together and become quite hard when allowed to stand for short periods of time. It has frequently to be blasted in the holds of ships and in storage warehouses.

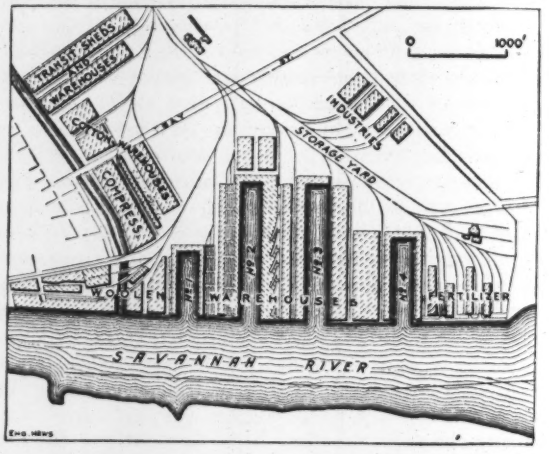


Fig. 1. General Layout of Terminal, Central of Georgia Ry. and Ocean Steamship Co. at Savannah, Ga.

The bagged material consists of various salts of potassium and kindred bases, imported from Europe, and nitrate of soda from Chile. This material is quite valuable; the nitrate, for instance, being worth about $55 per ton. The bags average in weight about 200 lb. From the very nature of the product these bags are usually badly weakened by rotting, which has proved to be a limitation in the speed of handling. The rate of acceleration incident to high hoisting speed causes failure of the bags; this limits the speed to approximately 175 ft. per min.

The material is valuable and of course must be protected from the weather. Although solid cargoes of bulk kainite are very often received, it most frequently comes in quite small lots of a few thousand tons only, which are separated in the ship and must be kept separate in the handling throughout, these lots being of different analysis or for different consignees. The usual loading consists of bulk material in the lower decks of each hatch and bags in the upper.

Ship Unloading

The mechanisms for handling free flowing freight, such as coal and ore, have reached a high degree of perfection, particularly in the lake traffic. These narrow waters permit of the design of ships specially for cheap cargo handling. Seaships, on the contrary, must be designed primarily to stand rough ocean voyages. They are divided by transverse bulkheads and each cargo

*Electrical Engineer, Central of Georgia Ry., Savannah, Ga.

space thus formed is divided by horizontal decks. The hatches are small and comparatively few in number. This renders a large percentage of the cargo area not accessible vertically from above.

The hardening of the bulk kainite in the hold requires that it be broken down and the most satisfactory method of handling it consists in the use of buckets, having a capacity of 20 cu.ft., and mounted on rollers, so as to permit being pulled into the space not directly under

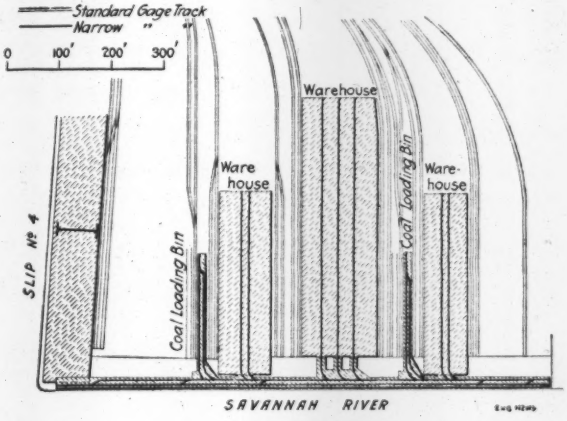


FIG. 2. DETAIL OF FERTILIZER HANDLING PLANT AT SAVANNAH

the hatches. This requires the use of hand labor in the hold. The only possibility of expedition consists in increased speed of hoisting. Under very favorable conditions, grab buckets may be used. For discharging the bagged material the ships' winches are used, because special machinery such as is employed in unloading solid cargoes, of cement, coffee, etc., is not warranted on account of the relatively small receipts, sorting of lots, the bad-season load factor, and other limitations, such as hoisting speed.

DELIVERY FROM SHIP-SIDE

It has been pointed out that kainite is valuable. and consequently must be protected from the weather and carefully weighed. It must be handled in numerous separate consignments, since the material coming from every hatch may be different. With two ships discharging at the same time, there may be as many as six separate consignments which must be kept entirely distinct throughout all handling. This is a condition entirely different from that found in the usual bulk handling plant, such as for coal and ore.

Much the greater part is handled direct from ship-side to box-cars. The remainder must be held by the terminal company in temporary storage in warehouses.

For handling the bulk material from ship-side the most suitable plan was found to be the use of elevated electric railway with individually driven motor cars. The general scheme of operation is to convey the material to be loaded into box-cars direct from the ship-side hoppers to a system of bins for temporary storage, each bin having a capacity of 40 tons, or one carload. The material for storage is conveyed into the inclosed warehouses and there dumped from elevated trestles.

These motor cars are operated upon a 24-in. gage track system, with a 30-lb. rail, supported on 4x8-in. crossties, spaced 24 in. An interesting phase of the layout is the use of spring switches, thus making the switching automatic. Two tracks are provided upon the waterfront trestle and the circulation of motor cars is downstream on the outer track and up on the inner. Numerous crossovers are provided so as to reduce the average length of the run and prevent interference, etc. The rear rail of the ship-unloading equipment is elevated so as to allow the motor cars to run under to the storage warehouses and the car-loading bins. The operation of this layout has been very satisfactory.

The motor cars are of the side-dumping type, having a load capacity of 5 tons and weighing empty 4 tons. The car body is supported through center pins on two so called maximum-traction trucks. These trucks have cast-steel side frames with rigid bolster; coil springs carry the weight to the axles; roller bearings are employed, principally for the reason that they do not require much attention. One of these cars is shown in Fig. 3. Each truck is equipped with a mine-type motor, geared to the axle; the wheel diameter is 20 in. and the gear ratio 4.77. The motors are connected in parallel, which gives a free running speed, loaded, on straight track of 10.4 miles per hour, corresponding to a tractive resistance of 21 lb. per ton and a power consumption of 68 watt-hours per ton-mile. The motor speed is 830 r.p.m. at the balancing speed of the car. Short-radius curves are, of course, used; namely, 20-, 25- and 50-ft. radius. Curve resistance is a variable quantity, depending upon speed, condition of track lubrication, etc. Measurements showed a tractive resistance of 60 lb. per ton on a curve of 25-ft. radius. A smaller radius than 20 ft. should not be used with cars of this type.

When dumping, particularly into the car-loading bins, or receiving from the ship-side hoppers, quantities of salt dust are thrown up. This penetrates the apparatus

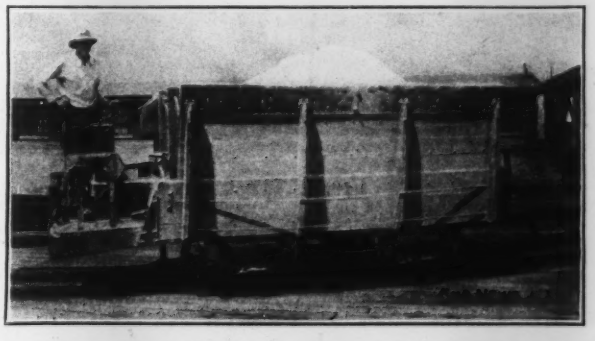


FIG. 3. MOTOR CAR USED IN HANDLING BULK KAINITE AT CENTRAL OF GEORGIA RY. TERMINAL, SAVANNAH

and then absorbs moisture. It is therefore absolutely necessary to use fully inclosed motors and to seal all outlets. The wiring too is very vulnerable—a good grade of rubber is satisfactory, but it is exceedingly difficult to prevent serious corrosion at the soldered joints—the use of lead cable is the best solution.

These cars are operated from a third-rail system. It was impossible to use trolleys on account of interfering with the overhead equipment. The third rail is a 20-lb. T-rail, supported every 10 ft. on special porcelain insulators, having large leakage distance. It was necessary

to use a low voltage on account of the formation of salt films in damp weather resulting in severe shocks from partial grounds.

Car Loading

It was desirable to load box-cars mechanically, principally so as to attain speed and also so as to reduce the number of men required and thus simplify the control for this intermittent service in both respects. The railway tracks were necessarily at right angles to the waterfront and stub ended. The general plan of car loading is to deliver from the overhead trestle into bins spaced 45 ft. c. to c. The material is fed from these bins to portable box-car loading machines which deliver it into the ends of the cars. A partial view of a set of bins is shown in Fig. 4. In order to reduce railway switching, arrangements were necessary so as to permit drilling only once or twice per working shift. This is accomplished by providing motor-operated hoists or car pullers. The empty cars are pulled in by means of these and the loaded ones pushed out.

Fig. 4. Trestle over Bins for Box-Car Loading of Bulk Kainite

The detail arrangements are as follows: The bins are designed to hold 40 tons, or a box-car load. The bin is in the form of an inverted pyramid and the material is fed out through a double-jaw cutoff valve having an opening of 24x30 in. There were misgivings as to whether kainite could be handled in this manner since it consolidates into a solid mass and has frequently to be blasted, both in ships and warehouses. It was found by experiment that this consolidation requires some hours and it seems to be due to the presence of moisture, induced by the salt. With the depth of the bin used, however, and the quick unloading of the bins, it was found that the opening above indicated gave free flow when combined with bin sides having an angle to horizontal of not less than 55°. On account of the limited height of the trestle, the maximum angle of chute which might be used to deliver from the discharge valve of the bin to the car-loading machine was 20°. At least 55° is required for free flow. It was therefore necessary to use mechanical feed. These chutes are built of sheet steel. They are given a reciprocating motion of 100 r.p.m. by means of a connecting-rod and crank having throw adjustment of 0 to 6 in. The feeder is directly driven by back-gear type motor, which is started and stopped by a quick break oil-switch. The spring control and oil immersion of this switch permit frequent starting and stopping with little wear; the operation is very satisfactory. The rate of feed with this equipment is 3 tons per minute.

The feeder delivers the kainite to car loader as shown in Fig. 5. The essential part of the loader is a belt 22 in. wide, driven at a speed of 850 ft. per min. Nothing but balata was found to be able to stand the action of the wet salt. The belt carriage is moved into the box-car by hand and can then be extended toward either end for a distance of approximately 10 ft.; it may also be elevated toward the roof of the car. The belt requires 10 hp.; it is driven by a series wound motor, having a nominal rating of 25 hp. The prime requisite was that the motor should be dust proof and extremely compact; no form of a.c. motor or shunt-wound d.c. motor was sufficiently compact; it was necessary to use a series motor. The choice as to size depends entirely on the heating, the cycle of the operation being such as to require this motor, though, of course, it is much stronger mechanically than necessary.

The whole machine is mounted on a four-wheel truck having 10-ft. gage and is driven on its tracks by a 50-hp. a.c. motor. This motor is used in switching empty cars and to assist in the spotting of cars opposite the bins. The belt speed is sufficient to load the ma-

terial into the ends of the cars; it both conveys and projects.

The placing of box-cars is accomplished as follows: A terminal-yard crew place a string of empties on one of the two tracks leading to the box-car loading bins. Under the end of the bin trestle is placed a double-drum car-puller driven by a 75-hp. motor; the ends of a 1-in. wire cable lying parallel to the track directly in front of the bins are connected to these drums; it passes around 42-in. diameter end sheaves and slack take up is provided. This cable is driven at a speed of 120 ft. per min. It was necessary to provide a tractive effort of approximately 25,000 lb. since a string of 16 to 20 loaded cars had to be pushed out round a 12° curve. This cable is attached to a pusher or barney; a short flat-car loaded with scrap iron. The necessary number of empties, three or more, are pulled in at a time from the empty track and then pushed out on the loaded track.

FIG. 5. BOX-CAR-LOADING MACHINE AT SAVANNAH TERMINAL

FIG. 6. STORAGE-BATTERY TRUCK FOR BOX-CAR LOADING

HANDLING BAG MATERIAL

As in the case of the bulk material, much the greater portion of the bagged must be delivered direct to the box-cars. With the stub-end arrangement of the railway tracks at right angles to the water front the haul from ship-side to cars is necessarily long. Quite a large part of this material has to be sorted for marks after being delivered from the ship, the consignments being more or less mixed in loading. In the case of solid cargoes, such as nitrate, which is very valuable, the weighing must be very carefully done and there must be considerable sampling. It is most desirable to make use of a mechanism which, at one handling, would take the material from ship-side and deliver it direct into the end of the box-car. Investigation showed the most promising machine to be the storage-battery truck.

The type of machine used is illustrated in Fig. 6. These machines are very flexible in operation; they will go practically anywhere a hand truck will go. The empty weight is 1900 lb. and the full-load capacity 4000 lb. The speed with full load and average battery voltage is 6.3 miles per hour and the free running speed light is 8.8 miles per hour. It is equipped with a 225-ampere-hour, 25-volt battery giving an output of 5.6 kw.-hr. It is really astonishing how effective this small amount of stored energy is in weight transportation. These machines will operate in regular service hauling average loads of 3000 lb. for 12 hours.

It is interesting to compare the performance of this type of vehicle with that of the narrow-gage motor cars above described. Fig. 7 shows the performance of a similar storage-battery truck having a light weight of 2600 lb. and a speed with full load of 7.5 miles per hour.

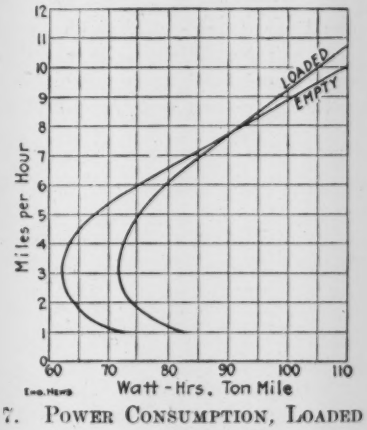


FIG. 7. POWER CONSUMPTION, LOADED AND EMPTY, OF MOTOR TRUCK

(Used in freight handling on ordinary brick pavement.)

The tractive resistance of the truck is approximately 34 lb. per ton at 7.5 miles per hour and that of the motor car on 24-in. gage track 21 lb. per ton at 10.4 miles per hour; similarly the watt-hours per ton-mile of the motor cars are 68 and of the truck 88, measuring the input at motor terminals; considering the efficiency of the battery this figure becomes 147. The radically different

type of motor will also be noted; the mine-type motor of the cars being a slow-speed machine, running at 830 r.p.m. at the full-load balancing car speed; the corresponding speed of the vehicle motor is 2200 r.p.m.

Electrically driven machinery is used throughout the plant. Power is purchased from the local power company at 2200 volts, 60 cycles and is transformed to 440 volts for the hoists and other large motors; and is converted to 220- and 110-volt direct current for use of the railway and for charging storage-battery trucks, etc. The electrical apparatus used in the plant was furnished by the Westinghouse Electric & Manufacturing Co., other equipment by the C. W. Hunt Co.

The work thus far completed was carried out by the railway company under the direction of C. K. Lawrence as chief engineer, H. F. Sharpely being principal assistant engineer, E. M. Rhett being in charge of mechanical and electrical layout and design. The work now under way at Slip No. 2 is being handled by the Ocean Steamship Co., through J. G. Basinger of New York.

Notes on Massachusetts Highway Work

SYNOPSIS—This article gives the results of interviews with engineers and a tour of inspection of the roads comprising the Boston district of the Massachusetts Highway Department. It gives a summary of the methods of survey and preliminary study, descriptions of the two most notable types of road surface developed in Massachusetts, maintenance experience, and an outline of the highway department organization.

Surveys and Preliminary Studies

The first survey is made so thorough that it answers all purposes of both preliminary study and permanent location. This survey is made by a headquarters corps reporting directly to the chief engineer.

The surveyed line or base line is run as close to the center-line of the highway as is consistent with all considerations. Stations are marked every 50 ft. with wedge shaped wood stakes 30 in. long (square stakes sawed diagonally) on both sides of the traveled way. Stone monuments are afterward set on both sides of the location lines at all P. C. and P. T. stations and at intermediate stations on long tangents.

All property lines, topography, and other data to be used in design are referred to this survey line, which is shown on the preliminary plans by a full red line. This survey line is always run so as to form a closed traverse by returning over a different route, thus giving a check on both distances and directions.

Plotting—From this survey line the center-line of the highway is offset and plotted in the office on the preliminary plans as a red dash line. Cross-sections are taken at the time of the original survey at every 50-ft. station for the whole width of the roadway. These cross-sections are plotted on a scale of 1 in. = 4 ft. on large sheets (24x36 in.) of cross-section paper in consecutive order, so that by a study of the completed plan and cross-sections, the center-line of the paved strip may be shifted to whatever part of the whole highway will give the best combination of grade, alignment, foundation, preservation of shade trees, etc. The profile of the center-line of the proposed improved road may then be worked out, and this is plotted on the same sheet with the plan (Fig. 1). It is always aimed to keep the new grade line as close as possible to the old one.

Division Engineer's Report—These plans, profiles, and cross-sections are then sent to the division engineer in whose division the improvement is projected. The division engineer studies the proposed changes in grade and alignment on the ground, and then makes a preliminary report to the chief engineer on a standard printed form. This report consists of six foolscap sheets, joined at the top, designed to be folded and filed, 4x8½ in. After noting the date of the receipt of the plans, and of the examination of the road, the division engineer fills in the following printed queries and explanations:

(1) Present road surface, character of.
(2) Surface sections and alignment, discussion of.
(Division engineers are expected to recommend the section of broken stone which will be most economical and satisfactory in each case. It is not necessary that the same section

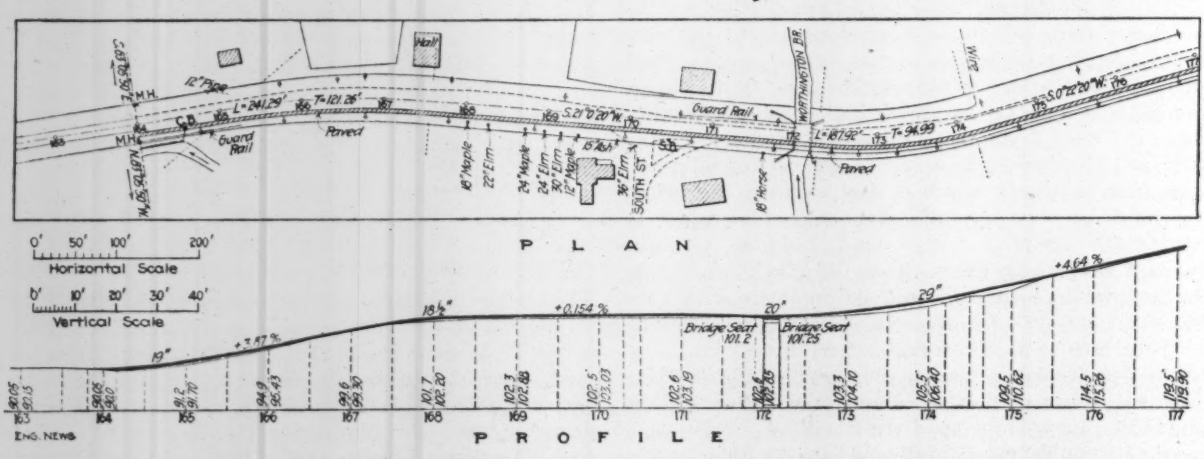


Fig. 1. Sample Road-Improvement Map and Profile, Massachusetts State Highway Commission

shall be adopted throughout the entire length of the road. When a change in section will result in a saving of stone by taking advantage of existing conditions, such a change should be recommended.)

(3) Location lines, discussion of.

(If changes can be made so as to avoid unnecessary land damages, division engineers are expected to recommend them.)

(4) Grade submitted, discussion of, with recommendations.

(5) Street railway tracks.

(Division engineers will state whether the present alignment and grade are satisfactory, and if not, what changes are necessary.)

(6) Borrow and gravel, material suitable for, location and quality of.

(Give sufficient data to permit approximate estimate of overhaul.

(7) Culverts, bridges and catchbasins.

(All structures with a span greater than 8 ft. will be classed as bridges. Division engineers will give such information as they can secure concerning drainage areas. When it appears to be necessary to obtain easements for the discharge of culverts on private land, mention should be made of the matter under this heading.)

(8) Bituminous treatment, discussion of.

(9) Foundations and drains, recommendations concerning.

(10) Guard rail, recommendations as to location of.

(11) Trees, value of, for shade.

(12) Ledge and other materials, not otherwise enumerated.

(13) Prices recommended for use in the preparation of the estimates and contracts. [Here follows a list of operations and materials with price per unit: Excavation, earth; excavation, borrow, not including overhaul; excavation, ledge; masonry; concrete, not including cement and steel; gravel; broken stone on cars; broken stone on roads; unloading stone; stone at crusher; breaking stone; teaming stone; spreading stone; watering stone; rolling stone; supervision; extra (3); total for broken stone; bituminous treatment, not including bitumen; various sizes of culvert pipe; guard rail; stone filling in place; cobblestone gutters; delivering and setting bounds; catchbasins, not including frames and grates.]

(14) Provisions for taking care of travel during construction.

(15) Remarks.

This scheme, as readily perceived, gives adequate information to the Commission and to the engineers in the main office, where specifications and contracts are drawn. Ample information regarding kind, character and method of improvement it is desirable to make is given so that final plans, profiles and specifications may be drawn that will properly fit the local conditions.

CHANGES IN ALIGNMENT—If a new right-of-way is required, it is referred to the town in which it occurs to act, or to the abutting property owners. The state does not directly condemn land for highway purposes until after the town or property owners have made agreements relative to land damage, as the improvement of the highway is a matter of local option, and the town must not only petition for the improvement, but smooth the way over all legal difficulties before the State Highway Commission will begin an improvement.

TYPES OF ROADS

Massachusetts has no hard and fast standard of road types. It thus happens that the state is still building many miles of water-bound macadam or gravel roads, which are cheapest, with the intention and expectation of converting them into bituminous macadam or oil gravel, whenever and wherever the traffic or repair conditions warrant the additional expenditure. In other instances, a bituminous macadam, asphaltic concrete, cement concrete, brick, stone block, or any kind of surfacing, may be used on such stretches of road as experience or the judgment of the engineers shows an economic necessity.

The plain macadam sections have an ordinary depth of surfacing of 4 in. or 6 in., with a flat-sided crown of ¾ in. to 1 ft. on 15 ft. cross-sections and ½ in. to 1 ft. on 18-ft. cross-sections. The 6-in. depth cross-section has a flat base, making the section 4 in. thick at the sides, while the base of 4-in. cross-section follows the crown of the surface, making it uniformly 4 in. thick. If the road is designed to be ultimately surfaced with a bituminous material the crown is made ½ in. to 1 ft. instead of ¾ in.

The bituminous-macadam surfaces are divided into three classes: penetration, surface spraying, and layer methods. In the penetration method the lower course is bound with stone screenings or sand and the upper course is No. 2 stone with a bituminous binder. In the surface-spraying method, the surface coat is of screenings or sand and bituminous binder. For the penetration method there are ordinarily two applications of asphaltic oil, each of ⅝ gal. per sq.yd. There may also be a third application of ¼ gal. per sq.yd. for a surface finish. For the surface-treatment method there is one application of ½ gal. per sq.yd., or two applications of ¼ gal. per sq.yd. on the finished surface of the roadway.

Another form of bituminous macadam is constructed by the so called layer method, which consists of five courses, as follows: One 2½-in. (2 in. at sides) course of 2½-in. stone filled with sand and rolled dry; a second course of ¾-in. to 1¼-in. stone, practically one stone thick, watered and covered with bitumen (not rolled); a third course of the same stone, practically one stone thick, watered and covered with bitumen and lightly rolled; a fourth layer of ⅛-in. to ½-in. pea stone, watered and covered with bitumen (not rolled); a fifth course of clean, coarse, gravelly sand $\frac{1}{16}$ in. to ¾ in. in diameter. This course is thoroughly rolled. This construction requires a total of 1 gal. of bitumen per sq.yd., spread on under a pressure of not less than 70 lb. per sq.in. at a temperature of about 200° F. A similar form of construction in four layers, omitting the second course of 1¼-in. stone, requires ½ gal. of bitumen per sq.yd.

SPECIAL TYPES

The scheme of improvement has naturally led to individual study and experiment to utilize local materials to the best possible advantage, resulting in at least two types of road which are, or at least have been, peculiar to Massachusetts: (1) the sand and oil, mixed, or gravel and oil, mixed; (2) asphaltic concrete by the penetration method.

GRAVEL AND OIL MIXED—The success of the sand and oil roads first constructed on Cape Cod in 1905 led to a continuation of this kind of construction. In 1910, the town of Concord laid down on the main street of Concord a pavement surface made of a mixture of gravel and hot asphaltic oil binder, the work being done under the direction of the Highway Commission's engineers, and this pavement is still in excellent condition (Figs. 2 and 3).

This type of construction is now being used on several roads where traffic does not warrant a bituminous macadam or a broken-stone asphaltic concrete, and where a good gravel is readily available and an untreated gravel would not stand. The bottom course is rolled gravel or broken field stones. The surfacing mixture is spread and rolled 2 in. thick, after the manner of sheet asphalt. After the rolling and surfacing are completed, a seal coat of asphaltic oil is distributed in two applications of ¼ gal. of oil per sq.yd. for each application. Each application of

FIG. 2. MAIN ST., CONCORD, MASS., PAVED WITH OIL-MIXED GRAVEL

FIG. 3. PAVEMENT SURFACE

oil is covered with a thin layer of sand or screened gravel and rerolled. Figs. 4 and 5 show the construction of such a road. The plant is simple and easily portable.

ASPHALTIC CONCRETE BY PENETRATION METHOD—A method of spraying a layer of broken stone with hot liquid asphalt makes a pavement which looks like sheet asphalt. As this type costs less than $1 per sq.yd., for a 3-in. surface, it is the cheapest asphalt surface yet constructed. A stretch of road in the town of Weston constructed by this method is in excellent condition after two years' service under quite heavy traffic.

The asphalt used was Bermudez road asphalt, which is a true asphalt, not cutback or emulsified. It was not possible to make this type of pavement until the invention of a spraying apparatus which could handle and distribute the heavier asphalts.

Some points of the specifications for this type of road are of interest. The lower course consists of stones passing a 2½-in. ring but retained on a 1¼-in. ring. The lower course is 4 in. deep when rolled. Upon this lower course is spread the upper course, consisting of stone of the same size as the lower course. This upper course is 2 to 3 in. thick after rolling, depending on traffic conditions.

Upon the upper course is uniformly distributed hot liquid asphalt at the rate of 1¾ gal. per sq.yd., and at once covered with a coating of clean pea stone, sufficient to fill the surface voids and permit a steam roller to pass over it without sticking to the asphalt. Brooms are used to spread this first coating of pea stone, and any surplus of stone is swept off the surface. A second application of asphalt is then made with a pressure distributor in such quantity as to completely coat the surface at the rate of not less than ⅓ gal. nor more than ½ gal. per sq.yd.

The second application must not be poured, but sprinkled or spread uniformly by means of a pressure distributor. After the second application of asphalt the surface is again covered with pea stone and again thoroughly rolled. The asphalt is applied at a temperature of approximately 300° F.

MAINTENANCE EXPERIENCE

The following notes in regard to maintenance were gleaned from an interview and inspection trip with F. C. Pillsbury, who has been division engineer of the Boston district for 19 years.

For the many miles of water-bound macadam and gravel roads, with which the state found itself at the be-

FIG. 4. CRUSHING AND SCREENING PLANT FOR OIL-MIXED GRAVEL ROAD

FIG. 5. CONSTRUCTION FIRST COURSE OF OIL-MIXED GRAVEL ROAD

ginning of the automobile traffic period, the best preservative has been found to be an annual application of a light cutback asphaltic oil. Tar preparations have often given the best results, but with tar the road needs more frequent treatment and more constant attention. The light asphaltic-oiled roads invariably become muddy for a few weeks in the spring, but when promptly shaped, crowned, rolled and given a light application of new oil (about $\frac{1}{5}$ gal. per sq.yd.), such roads remain in excellent condition for the remainder of the season.

FIG. 6. TYPE OF CONCRETE BRIDGE REPLACING OLD SINGLE-SPAN TRUSS

Roads treated with tar preparations are generally free from this mud nuisance, but on the other hand, the tar-treated road dries out quicker, cracks and disintegrates, i.e., loses its life and elasticity much sooner. Where roads could be constantly cared for and more frequently treated, tar is admitted the more satisfactory, but with a yearly or biennial treatment asphalt is preferred. Mixtures of tar and asphalt seem rather to have the bad qualities of both, so that true asphaltic oils are preferred.

These spring resurfacings and applications of oil are done as nearly simultaneously as possible throughout the whole state, both by the state's own forces and equipment and by contract with the different road-oil companies.

Besides this periodical maintenance work, a patrol system of repairing is in force. Barrels of oil are distributed at convenient places, and patrolmen are constantly making patches with a heavy asphaltic oil, which is covered with stone chips or gravel and sand.

Mr. Pillsbury recommended as probably one of the most reliable and useful tests for a bituminous road oil that for adhesiveness (described in ENGINEERING NEWS, Aug. 13, 1914, p. 343), by means of measuring the oil's power to prevent relative motion of two concentric cylinders. This has been found in many instances to be practically the sole test of the binding value of asphaltic oils and is considered by California chemists to be one of the most important tests.

As to maintenance and repair of pavement which is disturbed by subsurface construction, the state is amply protected by acquiring, when the road is taken over, absolute jurisdiction over all such subsurface structures. The division engineer has one assistant engineer whose sole duty it is to investigate petitions and issue permits for sewer, water-pipe, or other underground conduits, and to supervise repair work to insure that the pavement surface is fully restored.

By these methods the State of Massachusetts has maintained and kept in excellent condition many old road surfaces, representing a large original outlay, which under ordinary conditions would have rapidly disintegrated and have now been worse than useless. Such success have the Massachusetts highway engineers had in this work that they are by no means converted to the hard (brick or concrete) pavement theory. Indeed, it must be admitted that the roads in Massachusetts are hardly second to those of any state in the country, and this has been accomplished at a very modest cost, both for construction and maintenance.

HIGHWAY ORGANIZATION

The Highway Department as a whole is divided into two grand divisions under the Chief Engineer, A. W. Dean. The Office Division, under A. M. Lovis, has charge of surveys and designs. The Field Division consists of four territorial divisions, each in charge of a Division Engineer.

Division 1 comprises the west and southwest section of the state, and is in charge of Division Engineer J. A. Johnston, with headquarters at Springfield. Division 2, consisting of the north central portion of the state, is in charge of Division Engineer C. H. Howes, with headquarters at Greenfield. Division 3 comprises the territory in the vicinity of Boston, on the west and north, in the eastern part of the state, and is in charge of Division Engineer F. C. Pillsbury, with headquarters at Boston. Division 4 takes in the southeasterly portion of the state, including Cape Cod and the Islands, and is in charge of Division Engineer W. R. Farrington, with headquarters at Middleboro, Mass.

Each Division Engineer has a Principal Assistant Engineer. Each contract is in charge of a Resident Engineer, who reports to the Division Engineer or his principal assistant. Minor repairs are made by a patrol system under the supervision of an assistant engineer. A slight variation exists in the different divisions in minor points in the organization, such variation being deemed advisable in order to best suit the conditions of the division.

New Dry Dock for San Francisco

Two dry docks are now maintained on the Pacific Coast for the vessels of the United States Navy; one at Mare Island, on a branch of San Francisco Bay, the other at Bremerton on the Puget Sound near Seattle. With the opening of the Panama Canal, naval operations on the Pacific assume a much more important aspect than heretofore. Neither of the above docks will be adequate for the superdreadnaughts that will frequently need overhauling on the Coast. For this reason a contract has been executed between the United States and the Union Iron Works Dry Dock Co., of San Francisco, whereby

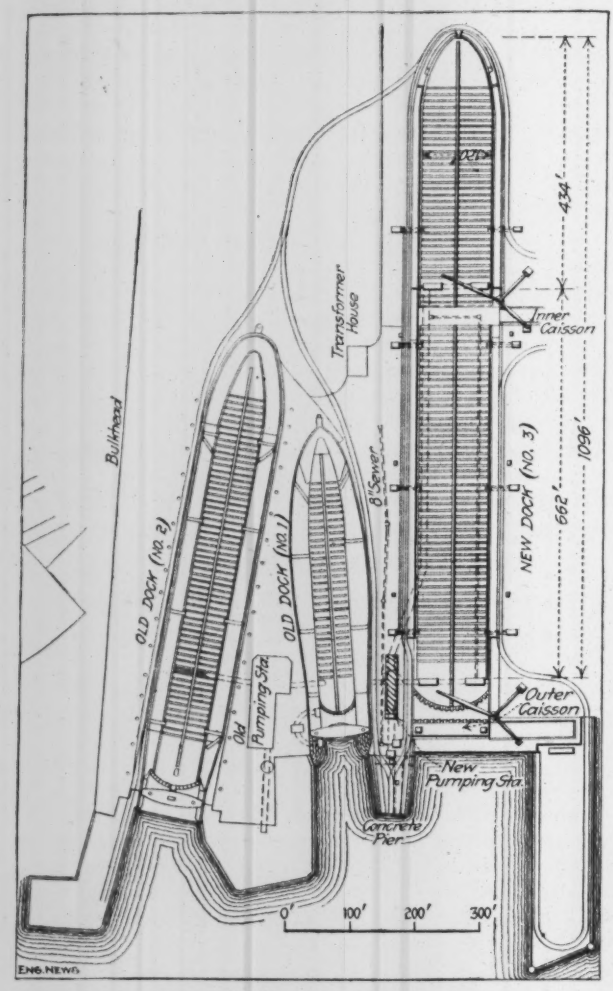


FIG. 1. LOCATION OF TWO OLD AND ONE NEW DRY DOCKS AT SAN FRANCISCO

the latter corporation will construct a $2,000,000 dry dock in San Francisco Harbor, under the guarantee that the government will furnish at least $50,000 in business each year for six successive years.

The new dock will be constructed at Hunter's Point, a promontory extending into the bay from the east shore of San Francisco, where two small dry docks have been in service for a number of years. The foundation will be of solid rock and the structure of reinforced concrete, specially designed to withstand severe earthquake shock. Its length over all is 1096 ft., breadth between vertical walls 120 ft., breadth to top of altars 140 ft. 8 in. and depth from mean high water to top of sill 42 ft. 6 in. Its bow is elliptical and its sides parallel. They arise vertically from the floor to within 12 ft. of the top of the coping, where altars extending for almost the entire length on both sides increase the width from 120 ft. to 140 ft. 8 in.

From the center to the side drainage-gutters the concrete floor has a chamber of 12 in. On it are the center keelson working platform and bilge blockways, both of wooden construction. Over the rudder pit, located near the outer end of the dock, the center keelson can be removed in sections.

The dock will be closed by an outer caisson and opened by floating and sliding this into a transverse recess in the side of the concrete structure. A similar recess and second caisson are located forward of the center, so that the dock may be used as a whole by sliding only the outer caisson into position; or the upper end alone may be used by closing the middle caisson, or both ends may be used independently at the same time by sliding closed both caissons. The center caisson recess is drained through a tunnel 4 ft. 6 in. in diameter, which leads to the pump pit. While the caissons themselves are essentially of the sliding-box type, they can also be floated from one place to another and thus divide the structure in compartments of any desired length. They are designed so as to have ample stability without fixed ballast both when floating light or when partially flooded. This is accomplished by transverse partitions which divide the caissons themselves into three compartments, the middle one of which can be flooded or emptied without lowering the metacenter below the center of gravity.

Four inclined tunnels 10 ft. in diameter, two on each side of the dock leading to the forward and rear sections, and two open-curved stairways at the elliptical bow, provide ready access to the interior. The tunnels are closed by butterfly valves. To unwater the dock these are opened and the water allowed to drain to the pump pit, from which it is by-drawn by 54-in. vertical centrifugal pumps of the single-stage, volute type. Each pump has a capacity of 75,000 gal. per min., pumping against a head up to 42 ft. They are direct connected to 750-hp. motors, which operate at 250 r.p.m. Pump casings are of the volute form, of cast iron, in sections provided with vertical and horizontal joints, so that they may be easily removed through openings in the motor floor above. Impellers of the inclosed type will be of a bronze with a single-bottom opening, and are designed to eliminate as far as possible objectionable end thrust. The top heads are sufficiently large to permit the removal of the impellers for repairs.

Pump house and pit are to be located near the outer end of the dry dock. The top of the pit will be 48 ft. below high water or 62 ft. 6 in. below the top of the dock coping. Besides the four 54-in. pumps for unwatering the docks, the equipment comprises two 15-in. pumps for draining the tunnels, an 8-in. salt-water high-pressure pump, a priming pump and the operating gear for the gate valves and for the butterfly valves which close the tunnels leading to each section of the dock.

For flooding the dock a tunnel 12 ft. in diameter, controlled by a butterfly valve, leads from the bay to the pump pit. Thence the water flows into the dock through

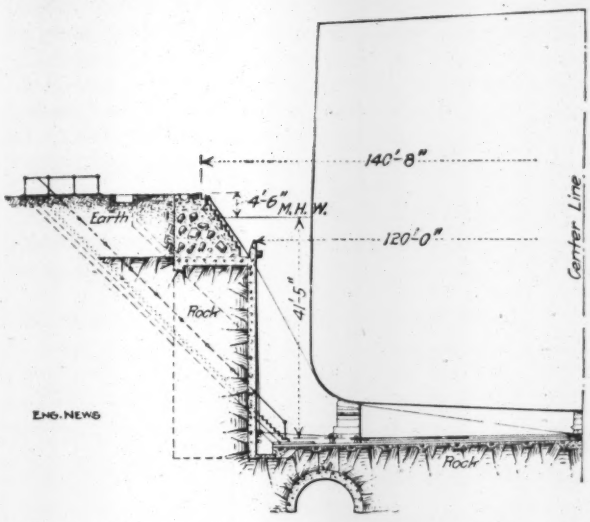


FIG. 2. HALF-SECTION THROUGH NEW SAN FRANCISCO DRY DOCK

the same tunnels by which the structure is unwatered. The operating gear which contains the flooding butterfly valve is housed separately from the main pumps in a small concrete building. The dock with tunnels has a capacity of 5,715,000 cu.ft. and with the four large pumps can be emptied in less than two hours and twenty minutes. It can be flooded in about thirty minutes.

On the port side of the dock will be a reinforced-concrete pier, with railroad tracks so arranged that cars may be landed from barges on the bay. A reinforced-concrete wharf on the starboard side approximately 450 ft. in length and 100 ft. in width at the outer end, will furnish ample wharfage facilities. Electric power will be used for the pumps and machinery as well as for the compressors that will operate pneumatic implements used in ship repair. A transformer house will therefore also be required. This will be of concrete and will be erected on the port side, opposite the inner caisson recess.

Below is a tabular description of the dock and caissons:

Length from outer meeting face to head of dock	1096 ft.	
Length from inner meeting face to head of dock	434 ft.	
Length from outer meeting face to inner meeting face	662 ft.	
Width of caissons	22 ft.	
Usable length of outer section of dock	640 ft.	
Usable length of inner section of dock	434 ft.	
Breadth between vertical walls	120 ft.	
Breadth at top in way of altars	140 ft.	8 in.
Height of coping above high water	4 ft.	6 in.
Outer sill below high water	40 ft.	
Depth from top of sill to top of coping	44 ft.	6 in.
Top of keelson below top of sill	1 ft.	5 in.
Height of keel blocks above top of keelson	4 ft.	
Top of keel block above top of sill	2 ft.	7 in.
Top of keel block below high water	37 ft.	5 in.
Flood camber		12 in.

	Inner caisson	Outer caisson
Length, molded	124 ft.	124 ft.
Breadth, molded	22 ft.	22 ft.
Extreme height at side	48 ft. 6 in.	46 ft. 6 in.
Light draft from third deck	8 ft. 6 in.	8 ft. 3 in.
Light displacement	620 tons	620 tons

Accessories included in the specifications are keel blocks, bilge blocks, air and salt-water piping, fresh-water

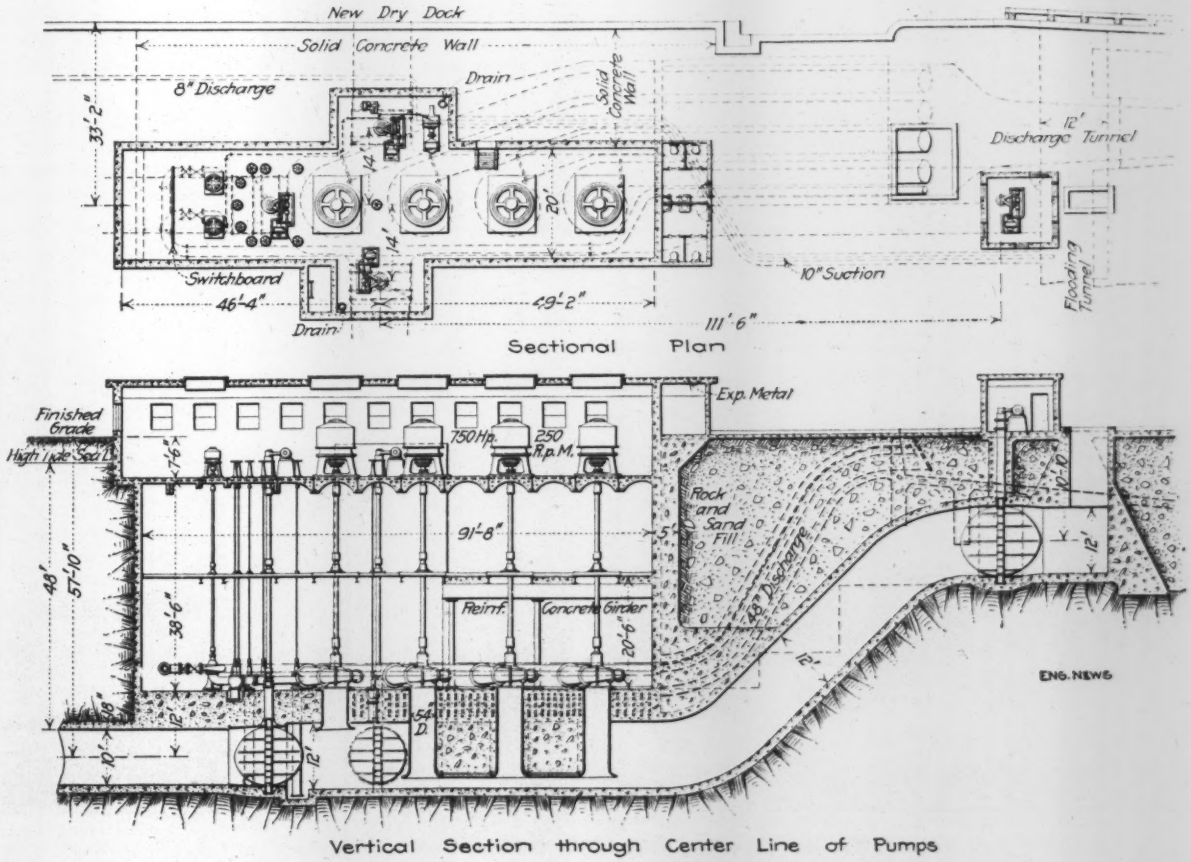


FIG. 3. SECTION THROUGH AND PLAN OF PUMP HOUSE, SAN FRANCISCO DRY DOCK

hydrants, electric conduits, electric capstans, one 50-ton electric self-slewing stiff-leg traveling crane, cast-iron bitts, belaying pins, ladders, blocks, etc.

The dock was designed and specifications prepared by Hugo P. Frear, Architect for the Union Iron Works Co. and Chief Engineer for the Union Iron Works Dry Dock Company.

The new San Francisco Dry Dock is among the five new dry docks which have a total length of 1000 ft. or over, all of which can dock any vessel now afloat. Two of these docks, the Gladstone at Liverpool and the Alexandria at Bombay, are completed and those at Boston and Levis are now under construction. The general dimensions of the five are as follows:

	Length ft.	Breadth ft.	Depth H.W. ft.
Boston, Mass.	1200	120	44.8
Levis, Que.	1120	120	40.0
San Francisco, Calif.	1096	120	42.5
Gladstone, Liverpool	1050	120	46.0
Bombay, India	1000	100	33.3

Bulk-Cement Handling Plant, Northwest Station, Commonwealth Edison Co., Chicago

By Harold W. Robbins*

A large concreting plant handling cement in bulk is of particular interest in construction work, especially as considered in connection with the building of one of the greatest of modern power stations, the Northwest Station of the Commonwealth Edison Co., Chicago, which is planned to have ultimately a capacity of over one-third of a million horsepower. Northwest Station is projected to supply in an increasing degree the power and lighting requirements of the adjacent part of Chicago, and to supplement the power stations on the south branch of the river at Fisk St. and Quarry St., which have respectively 165,000- and 84,000-kw. capacity.

Prominent features in this construction work are, the increasing of the floor area to approximately 100,000 sq.ft. and the making provision for four new units, at least, of 30,000 kw. each, which, when they are installed, eventually will bring the total capacity of this part of the power station up to not less than 160,000 kw. Besides the 50,000 cu.yd. of excavation, there are involved 18,000 cu.yd. of concrete, 3500 tons of steelwork and 200,000 rivets. A maximum of about 400 men have been employed on the construction work, and these, with the additional men working for the subcontractors, would probably total about 700 men engaged in finishing the job in record time.

The arrangement by which the 20,000 bbl. of cement, which probably will be required for the concreting work in various sections of the station, are handled in bulk through the central concreting plant is very interesting. This type of construction equipment is not only one of the newer developments for work of this kind, but in addition to its comparatively greater capacity also permits the very efficient production of concrete.

In a general way, it is similar to the one made use of in building the foundations of the new Baldwin Locomotive Works, at East Chicago, Ind., in 1913, where over 10,000 cu.yd. of concrete were placed. The proportionately greater requirements at Northwest Station made advisable the erection of this larger concreting plant, whose general dimensions are approximately 50 ft. length, 38 ft. height over all and 15 ft. width. It comprises essentially elevated storage bins for cement, sand and crushed stone, a bucket elevator, a power scraper for unloading cement from bulk cement cars, a derrick for transferring sand and stone and a batch concrete mixer. The machinery is all electrically driven by connection with the 220-volt alternating current distributed for general use in the yards of the power station.

Description of Plant

The storage bins are large and designed to be ample in supplying material for an average day's requirements.

*Universal Portland Cement Co., 208 So. La Salle St., Chicago, Ill.

Fig. 1. Loading Side of Concrete-Mixing Plant

(Cement brought in on cars in bulk, shoveled into elevator and carried to overhead bins. Sand and stone brought in on cars and lifted by derrick to overhead bins.)

The sand-storage bin has a volume of 1260 cu.ft. and the stone-storage bin 2300 cu.ft. The bulk-cement storage bin, located between the other two, and entirely inclosed against the weather, will hold a volume of nearly 1400 cu.ft., or 350 bbl. The unusually great weight of the

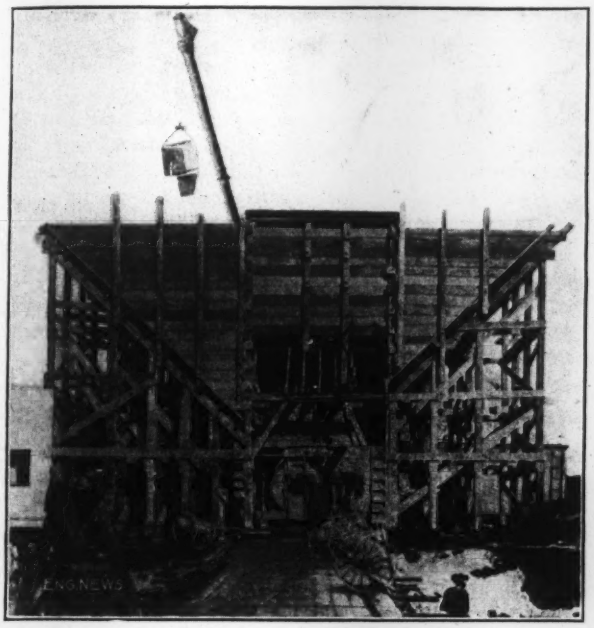


FIG. 2. DELIVERY SIDE OF CONCRETE-MIXING PLANT, COMMONWEALTH EDISON NORTHWEST STATION
(Concrete from mixer wheeled in buggies to job.)

material, supported at the elevation of 25 or 30 ft., necessitates heavy timbering for safety, and this is well taken care of by the use of 40 or more 10x10-in. supporting posts, adequately braced with diagonal timbers. As originally designed, the heavy stresses on the sides and bottoms of the storage bins were to be carried by similar 10x10-in. timbers, but it was found practicable to use steel tie-rods instead in connection with the 2x12-in. plank used for the sheathing of the bins, thus appreciably reducing the weight of the elevated sections.

In transferring bulk cement the methods usual in grain handling are employed. As the carloads of cement are received from the Buffington mills and spotted on the side track adjacent to the handling plant, the tapered chute shown in Fig. 3 is connected between the side opening of the box-car and the pit at the bottom of the elevator leg, a distance of 5 ft., and at an angle of about 45°. The chute is 9 ft. wide at the top and tapers down to about 4 ft. at the bottom. It is hinged at its lower end so that it may be swung up vertically out of the way when the cement cars are to be shifted on the side track. When resting against the side of the car, it comes just below the floor level and a canvas slip is fitted over the car edge to facilitate chuting the cement to the adjacent elevator leg.

Bulk cement is unloaded by means of a small steel scraper about 2 ft. wide, power operated, but guided inside of the car by one or two laborers. A 1-in. hemp rope leads from the scraper and around suitable blocks up to the windlass on the "shovel shaft," which is 10 ft. above the ground. Steel-spring shock absorbers connected to one of the blocks take the strain when the scraper is started with its load or in the event of its striking any irregularities on the car floor. The clutch engaging the windlass on the shovel shaft is operated by the men inside the cement car by means of a cord. The shovel shaft is gear driven from an adjacent countershaft.

The elevator (Fig. 1) is a standard type made by the Dodge Manufacturing Co., Mishawaka, Ind., and has a capacity for handling one carload of cement per hour. It is inclosed in a self-supporting steel casing, 38½ ft. in height over all, and is approximately 4x1¾ ft. in cross-section. Cement, on passing in through the grating to the bottom of the leg, is picked up by 6x6x12-in. steel buckets and discharged 35 ft. above, into the storage bin, which has a capacity of about 1½ cars. Drive is by belt from the 15-hp. motor to the countershaft, 12 ft. away. The latter is carried on wall brackets bolted to the upright timbers supporting the bins.

About 50 ft. to the north of the storage bins and with two side tracks intervening is a derrick with 80-ft. boom, by which the aggregates are transferred into the sand and stone bins, which are open to the sky. The hoisting apparatus used in raising the aggregates to the 30-ft. elevation includes an equipment of five power drums

FIG. 3. CHUTING BULK CEMENT FROM CAR TO ELEVATOR

operated by a 50-hp. motor. These drums control the movements of the Kieseler one-yard clamshell bucket in transferring the material from the gondolas to the storage bins or stockpiles on the ground and move the cars as they are unloaded.

In operation the bucket was observed to make a round trip from the gondola to the elevated storage bins in 40

seconds, which would permit the unloading of a 40-yd. car in from one-half to three-quarters of an hour. Ordinarily, eight cars per day are found to be handled easily. The crushed-stone stockpile usually has in reserve about 25 carloads and the sand stockpile contains about half of that number of loads.

Bulk cement is discharged from the storage bin through a vertical chute into the mixing hopper. In so doing it passes through a sheet-steel measuring section with gates at the top and bottom. One of three $\frac{3}{16}$-in. holes in this section is used to gage the amount required for a given mix —that is, for the usual run or for finishing mortar. The measured cement is released through a hinged door at the bottom of the chute by the operator on the charging platform.

Sand and stone are admitted through oscillating gates in the open steel chutes slanted at an angle of 45° from the bottom of the storage bins into the 40-ft. capacity charging hopper. The opening in front of the charging hopper is covered by a canvas apron, while the cement, sand and stone are being admitted, and this is found to effectually prevent the escape of any dust.

Exit from the charging hopper is controlled by a gate operated also from the charging platform above it (Fig. 2). The water supply is arranged to be automatically sprayed into the batch as it is mixing, and to be admitted in larger quantity when the material passes from the hopper into the mixer. The 1¼-cu.yd. Marsh-Capron batch mixer is driven by a 20-hp. motor. In practice here it has been called upon to produce batches of 33 cu.ft. oftener than 40 times within an hour.

Concrete is wheeled to various parts of the work in Sterling buggies carrying as a rule 5 cu.ft. each. Six or seven loads are made of each batch turned out by the mixer. At times there have been the full lot of 18 or 20 buggies in operation, but the production of concrete is varied directly as there is opportunity to place it in various parts of the building.

Performance

Besides the wheelers and outside men, those actually engaged in the concreting plant are:

On charging platform	2 men
Dumping mixer	1
Unloading bulk cement	2 (half time)
Trimming sand and stone cars	2
Hoisting engineer	1
Foreman	1

These are exclusive of the inspector employed to check the proportions of the mix and to insure uniformity.

In handling cement in bulk no labor is required on the charging platform to empty or remove sacks. In unloading a car two men in four hours or less can shovel out 250 or more barrels, using the power apparatus. In the matter of economy there is then the saving of the labor of one man in emptying sacks and of another in tying, bundling and taking them away, while the unloading from the car requires no extra men.

The maximum concrete placed in an 8-hour day has been slightly over 230 cu.yd. and the average that has been required in a 7-hour run has been 190 to 200 yd. This output is limited by the ability to use the concrete at the various points required and is a fraction of the full output of which the concreting plant is capable. In constructing the plant it was figured for a maximum duty of one batch a minute from a one-yard mixer. Those in charge estimate that it should be easily possible to turn out 60 batches, 1¼ yd. each, in an hour with this equipment, which would mean a possible total of over 500 cu.yd. in a day's run. The capacity of the storage bins is enough for 100 batches at least without renewal and the maximum rate of output would be easily obtainable by reason of the adequate facilities for unloading bulk cement and for obtaining sand and stone from the adjacent stockpiles or from supply cars on the siding.

The architectural design of the Northwest Station for the Commonwealth Edison Co. is by Holabird & Roche, Chicago. The steam-engineering work was laid out by Sargent & Lundy, Chicago, and the electrical work by the staff of the Commonwealth Edison Co. The Thompson-Starrett Co., of Chicago, with Fred D. Lyon in active superintendence, is in charge of the building construction and the central concreting plant was designed following the ideas of W. L. Chandler, of the Dodge Manufacturing Co., Mishawaka, Ind. The cement is shipped in carload lots from the mills of the Universal Portland Cement Co., at Buffington, Ind., a distance of 30 miles.

New Formulas for the Flow of Fluids in Pipes

By A. H. Anderson*

There is great uncertainty regarding the formula for the flow of fluids in pipes. The following gives a discussion of the subject.

Darcy, in 1857, found that Chezy's formula

$$f \times \frac{v^2}{2g} = \frac{d}{4} \times \frac{h}{l}$$

could be used only with a coefficient f varying with the diameter of the pipe. Engineers dealing with water have used Darcy's results, while gas manufacturers have always used Dr. Pole's formula, which is

$$Q = 1350 \sqrt{\frac{HD^5}{WL}}$$

being a solution of Chezy's formula for the given units with a value of $f = 0.006$.

Unwin, in 1904, proposed that the value of the Chezy coefficient should be

$$f = a\left(1 + \frac{b}{d}\right)$$

where $a = 0.0041$; $b = 0.143$; d = diameter of the pipe in feet. It will be noticed that the coefficient decreases as the diameter increases.

In a paper entitled "Flow of Gas Formulas," read before the Illinois Gas Association in 1912, J. M. Spitzglass, of Chicago, has shown by extensive tests that the coefficient of friction has a minimum value for a 12-in. pipe. For smaller pipes, the coefficient increases rapidly; for larger pipes than 12 in., the coefficient also increases, but not so rapidly. This was explained by the fact that loss of head is not entirely due to skin friction between the fluid and the wall of the pipe, but also due to the relative motions in the mass of the fluid or to the internal friction of the flow. This was also demonstrated by Prof. Osborne Reynolds (Philosophical Transactions, 1883).

Considering skin friction alone, the head lost will diminish as the diameter increases. Considering the in-

*Assistant Professor Mechanical Engineering, Armour Institute of Technology, Chicago, Ill.

ternal friction, the head lost increases with the cross-sectional area of the pipe. The internal friction affects the coefficient to a smaller extent than does the skin friction, but the internal friction being in proportion with the square of the diameter, the resultant coefficient reaches a point where, instead of diminishing, it slowly increases with an increase of diameter. This is borne out by the tests of Mr. Spitzglass.

The tests were made on 1-, 1½-, 2-, 3-, 24- and 36-in. pipes for low-pressure gas, and on 10-, 16- and 24-in. pipes for high-pressure gas. These experiments related only to illuminating gas and have been accepted in the gas field as authoritative. However, it is quite certain that the coefficient of friction will vary as stated, no matter what fluid is considered. This is corroborated by the fact that the fundamental hydraulic formula applies to any fluid and the coefficient of friction is taken the same for gas, water, air or steam. Therefore the writer feels justified in assuming that the coefficient of friction, depending upon the diameter, will vary the same way for all other fluids as it does for gas. From the data of the tests a new expression for the coefficient of friction was derived, as follows:

$$f = a\left(1 + \frac{b}{d} + cd\right)$$

where $a = 0.00315$; $b = 3.6$; $c = 0.03$; d = diameter of the pipe in inches.

The following table shows the deviation of Pole's and Unwin's formulas from the Spitzglass formula:

DISCHARGE IN PER CENT. OF SPITZGLASS' FORMULA

Diam. of pipe, in.	By Pole's formula	By Unwin's formula
1	150	111
1¼	135	109
1½	128	107
2	118	108
2½	112.5	104
3	108	102.5
3½	103	102
4	100	101.5
5	97	100.5
6	94.5	100.5
8	92	100.5
10	91	101
12	91	102
14	91.5	104
16	92.5	105.5
20	94.5	108
24	97.5	112
30	100.5	117.5
36	104.5	122.5
42	108.5	127.5

Formulas for Various Cases*

The expression for the quantity of discharge (derived from Chezy's formula) for any fluid has the form

$$q = 3.15\sqrt{\frac{1}{f}}\sqrt{\frac{p}{y}\cdot\frac{D^5}{l}}$$

where

q = discharge, cu.ft. per sec.;
p = friction drop, lb. per sq.ft.;
y = weight of fluid, lb. per cu.ft.;
D = diameter of pipe ft.;
l = length of pipe, ft.;
f = coefficient of friction.

Low-Pressure Air or Gas—Substitute the value of f as determined from Spitzglass' tests, and the formula becomes, for low-pressure air or gas,

$$Q = 3335\sqrt{\frac{Hd^5}{wl\left(1 + \frac{3.6}{d} + 0.03\,d\right)}}$$

where

Q = discharge, cu.ft. per hr.;
H = friction drop, in. of water;
w = density of the fluid referred to air as unity;
d = diameter of pipe, in.;
l = length of the pipe, ft.

High-Pressure Air or Gas—The equation becomes

$$Q = 64.3\sqrt{\frac{pAd^5}{wl\left(1 + \frac{3.6}{d} + 0.03\,d\right)}}$$

where

Q = discharge, cu.ft., free air or gas per min.;
p = friction drop, lb. per sq.in.;
A = average pressure in pipe line, lb. per sq.in. absolute;
d = diameter of pipe, in.;
l = length of pipe, ft.;
w = density of the fluid referred to air as unity.

Steam—For the flow of steam the equation becomes

$$W = 80\sqrt{\frac{pyd^5}{l\left(1 + \frac{3.6}{d} + 0.03\,d\right)}}$$

where

W = discharge, lb. per min.;
p = friction drop, lb. per sq.in.;
y = density of steam at mean pressure;
d = diameter of pipe, in.;
l = length of pipe, ft.

Water—For the flow of water the equation becomes

$$G = 50\sqrt{\frac{H'd^5}{l\left(1 + \frac{3.6}{d} + 0.03\,d\right)}}$$

where

G = discharge, gal. per min.;
H' = friction drop, ft. of water;
d = diameter of pipe, in.;
l = length of pipe, ft.

The writer has developed a slide-rule giving the solution of all these formulas.

*Mr. Spitzglass has developed a slide-rule giving the solution of all the formulas cited below.

Track Elevation at Milwaukee, Wis., is being extended north from the Kinnickinnic River to the Milwaukee River under an order of the Wisconsin Railway Commission. The work of the Chicago & Northwestern Ry. includes 1.04 miles of the Wisconsin Division, and 0.77-mile of the Madison Division from Kinnickinnic Ave. north to the junction with the Wisconsin Division. This includes 3.6 miles of main tracks and 1.4 of side tracks, while yard tracks make the total 14.80 miles of single track. There will be three subways at Greenfield, Washington and National Ave. Two railway grade crossings will be maintained at the new elevation, and three interlocking plants installed. The station at Allis has been raised, the new basement being used as a temporary station until the tracks are raised. The underground and foundation work is finished, sand filling is in progress, and concrete deck slabs for the subways have been made and stored ready for erection in the spring. The total estimated cost is $1,225,000, and the work is about 30% completed.

The work on the Chicago, Milwaukee & St. Paul Ry. extends from Kinnickinnic Ave. north to Fowler St. and from Clinton St. west to First Ave. The distance is about 1.4 miles, with 14 grade crossings eliminated, and a new station at Allis (National Ave.) will be required. This work will bring the track elevation into the center of the commercial district and within a few blocks of the union station. The piling is being driven, the underground work and foundations built, and the retaining walls built. The work is about 20% completed.

New York Rapid Transit Railway Extensions*

By F. Lavis†

VII--Methods of Timbering to Support the Street Surface on New York Subway Construction

The specifications for the construction of the subways in Manhattan and in most of the streets in Brooklyn require that the work be "carried on under covered roadways." This practically means that the paved surface of the street

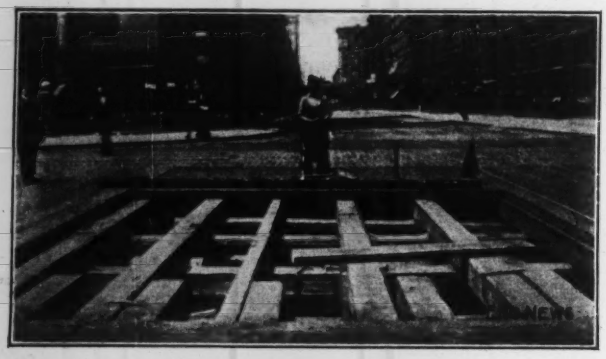


FIG. 34. TYPICAL TIMBER DECKING OVER SUBWAY EXCAVATION, SHOWING STRINGERS SUPPORTING STREET PLANKING

and generally also of the sidewalks has to be taken up and replaced by a timber deck, under which the excavation and construction may proceed with little or no interruption of the ordinary street traffic. Openings for shafts to give access to the excavations are permitted at intervals of 300 to 500 ft. in the upper part of the city, but are about 1000 ft. apart in the lower section.

In nearly all the streets in Manhattan where the subways are being or are to be built, there is a double-track street railway with underground contact system which has to be supported. In Brooklyn, the Bronx and Queens, the overhead trolley is used, which makes the problem of track support somewhat easier, though of course the poles have to be taken care of.

The usual method of procedure is first to excavate about 3 ft. of the street surface on one side of the tracks, putting in the decking and track supports in the form shown in the accompanying views. When one side of the street is decked over, the other side is taken care of in the same way.

Excavation is then carried on under this decking, the first lift being from 10 to 15 ft. in depth, practically the depth of the ordinary cellars and basements, the walls of which usually form the sides of the excavation. In very wide streets the full width is, of course, not taken out, but where the additional width beyond the neat lines is not excessive, the whole width is excavated in this first lift, as this permits easy access to the buildings for the underpinning operations. Where the full width to the cellar or vault walls at the sides is taken out, no sheeting is required on this first lift, but if this is not done sheeting must necessarily be driven from the surface.

Below this first lift, the ordinary form of timbering, using rangers and braces (see Fig. 35), may be continued in much the same manner as for the excavation of any trench, though, of course, on a larger scale, or one of the many special forms hereinafter described and illustrated may be used. There are two general types, one for earth and one for rock, the former well illustrated on the left and the latter on the right in Fig. 35 and in Fig. 38 and the accompanying drawings.

The essential differences are the necessity in earth excavation of supporting the side sheeting as well as the decking and in rock the provision of a clear working space and to guard against disaster by the loosening or destruction of one or more supports by the blasting oper-

*Copyright, 1914, by F. Lavis.

†Consulting Engineer, 50 Church St., New York City.

FIG. 35. TIMBERING IN EARTH EXCAVATION AT LEFT AND ROAD EXCAVATION AT RIGHT, ON LEXINGTON AVE.

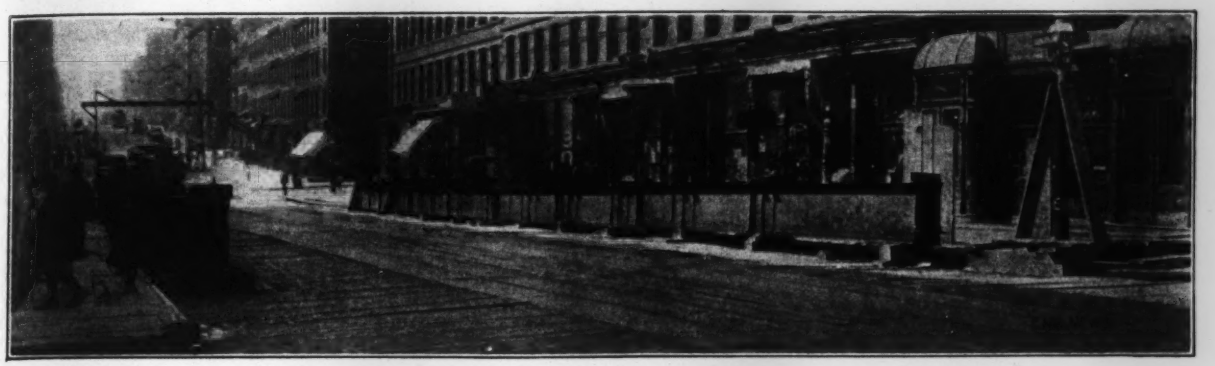


FIG. 36. STEEL GIRDERS ON STREET SURFACE CARRYING TIMBERING AND DECKING, LEXINGTON AVE., SECTION 11, ROUTE 5

ations or by slides in the very unstable New York rock.

One of the most interesting of the methods used for support in a deep rock excavation is that developed on Section 13, Lexington Ave., by Messrs. McMullen, Snare & Triest and illustrated quite clearly in the photograph, Fig. 38, and the sketch, Fig. 39.

On this section the concrete troughs which support the street-railway tracks are first supported longitudinally by the three or more 6x12-in. timbers laid flat (more than three when there are ducts to be taken care of). These, as the drawing, Fig. 39, shows, are held up by 12x12-in. cross-beams 18 ft. long, which are blocked up from the "needle beams" *F*, which are 12-in. 31½-lb. I-beams 30 ft. long spaced 10 ft. apart. It may be noted that it was not usually possible to put these needle beams directly under the blocking of the troughs of the street-railway tracks, on account of the presence of various gas and water pipes, etc., at about that level, a condition which obtains quite generally.

These needle beams have two pairs of 6x12-in. yellow-pine blocks about 5 ft. long bolted to them, one on each side and spaced so that they will come directly under the tracks, as shown on the sketch. These wooden blocks have their corners cut away so that they fit tight against the web and under the flanges of the I-beams, making at these blocks solid points of support for the longitudinal I-beam stringers underneath or for any temporary blocking or posts which may be required, and tending to prevent any overturning of the needle beam. Long X-braces and turnbuckles are also used between the needles.

At the end of the needle beam, holes are drilled so that 6x12-in. struts to the sides can be bolted to it, the 6x12's being fitted tight to the I-beam the same as the needle blocking.

The needle beams are then supported on the timber towers shown in the photograph, Fig. 38, by two pairs of 20-in. 65-lb. I-beams (*A* in Fig. 39), which are bolted together by long plates to develop full strength at the joints, making them equivalent to a continuous beam the whole length of the work. On either side of these two pairs of what might be called permanent longitudinal supports, are two pairs of the same size I-beams, bolted together the same way, the outer ones *C* 80 ft. long, and the inner ones *B* 120 ft. long, these latter being used as supports from the last timber tower over the face of the excavation to give a clear span of 50 to 60 ft. over the working space.

The inside 120-ft. pair is supported on the tower and on blocking just back of the working face, but also projects back of the tower and beyond the blocking, and these overhanging portions are wedged down tight from the decking, making it act as a cantilever (see sketch, Fig. 39). The 80-ft. pair spans from the tower to blocking ahead of the working face.

The towers are spaced 40 ft. c. to c., and as the excavation progresses, and space is cleared for a new tower,

FIG. 37. STREET DECKING AND TIMBERING CARRIED ON HOLLOW STEEL PILES UNDER CHURCH ST., SECTION 1, ROUTE 5

these pairs of I-beams are moved ahead for another space.

In the timbering to support the street decking and the electric-car tracks the plan adopted on Section 9, giving continuous support to the street-car tracks by means of I-beams spliced so as to develop full strength, is worthy of note. As shown in Fig. 40, there are three pairs of these beams directly under the decking. The cross-timber on which the tracks are supported is suspended from the I-beams. The side struts or diagonals also give additional arching support so that the danger due to the displacement of any of the posts in the excavation is reduced to a minimum. Fig. 40 shows an effective method of obtaining clear support over the rock excavation for the construction of the lower-level tracks.

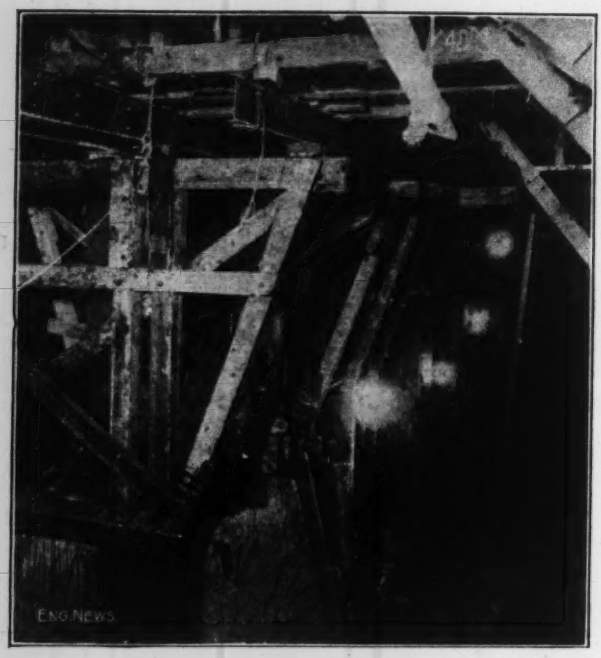


FIG. 38. TOWER SYSTEM OF TIMBERING IN ROCK EXCAVATION, LEXINGTON AVE., SECTION 13

On Sections 10 and 11, after some trouble with slipping and sliding rock, an additional means of supporting the street decking was adopted. Continuous girders about 4 ft. high and 150 to 200 ft. long were erected on top of the decking near the edge of the sidewalk and the timbering was virtually suspended from them, as is shown in Fig. 41, and the photograph, Fig. 36. These girders were generally intended to be supplementary to the system of supports beneath the deck and to be necessary only in case of the displacement of these latter. As a matter of fact, however, they proved useful in spanning the spaces where work was actually being carried on where changes and replacements of these lower supports were frequently necessary. Their disadvantage is that they occupy space on the street surface.

Where the regular system of cross-bracing and rangers are required, as in Section 14, Lexington Ave. (McMullen & Hoff contractors), where there is earth nearly to subgrade, some very elaborate systems of timbering are necessary. The type of timber construction on this section is shown in the photographs, Fig. 35; there are seven sets of braces in a depth of about 40 ft. below the

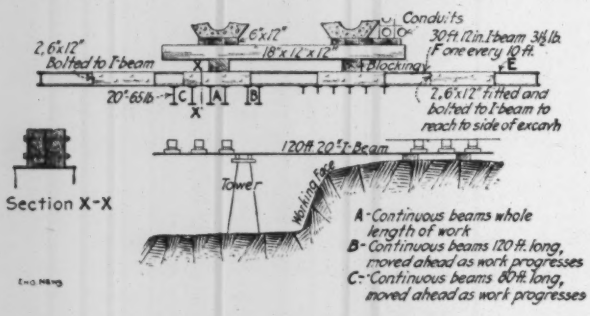


FIG. 39. SKETCH SHOWING STEEL BEAM SUPPORTS FOR TRACKS OVER ROCK EXCAVATION, LEXINGTON AVE., SECTION 13.

deck and the bents are about 10 ft. apart. Tension rods are put in to help hold up the bottom braces during excavation. The sheeting on this section is mostly Lackawanna steel, 14-in. arched-web, 41-lb., but some 6-in. plank (plain) is used. The steel piling mostly showed up quite well, although there were numerous boulders; and, of course, in some places where they were struck, the piling was more or less out of line toward the bottom.

On this section careful additional horizontal cross-bracing was carried diagonally from the center at each of the shafts through several bents to the sides, which added materially to the rigidity of the whole structure.

On the down-town sections, nearly all of which are in earth, more or less similar types of timbering are used. The sides are necessarily held by sheeting, generally wood, and the timbering is the usual system of rangers, braces and posts, though varied in detail by each of the several contractors.

The street decking is, of course, laid first, in some cases with a heavy wearing surface of 6-in. planks, in others 2-in. planks on closely spaced 6x6-in. timbers or 6-in. I-beams. The first lift of the excavation is then taken out to a depth of 10 or 15 ft., the street-railway tracks and pipes secured and generally one brace is carried across the whole width of the excavation, this first long brace being usually not less than 5 or 6 ft. below the surface. Great care, of course, is taken in all these sections to arrange the timbers so that they will clear the steel when the latter is erected.

The two principal variations seem to be in making either the posts or the braces continuous. On Section 2, the first operation was to sink the posts in sheeted pits to subgrade. The material was sand, and many men in New York now have become quite expert in sinking these 4x4-ft. pits in soft material by the use of horizontal sheeting (which will be described in more detail under the head of underpinning), so that the sinking of this large number of pits is not so serious an operation as might be imagined. No water, of moment, was encountered on this section above subgrade. Once the posts were down, of

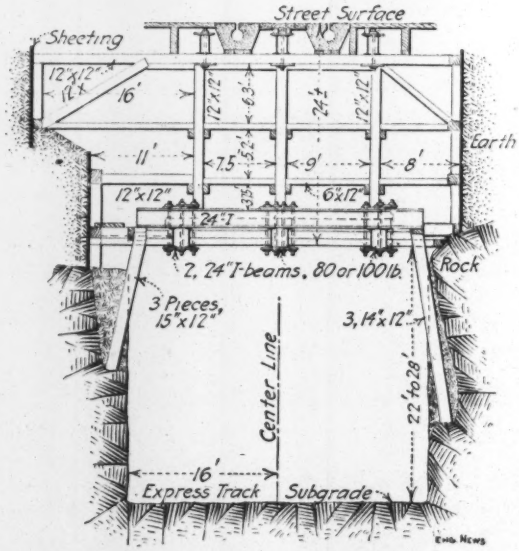


FIG 40. TIMBERING FOR ROCK EXCAVATION FOR DOUBLE-DECK SUBWAY, LEXINGTON AVE., BETWEEN 78TH AND 79TH ST., SECTION 9.

course, the deck was held in safety for any operation and the continuance of the excavation and timbering was a matter of routine.

A method used on part of Section 3 is clearly shown in Fig. 42, which needs little explanation. The building of the concrete sidewall was not followed throughout the section. The method of framing the timbers to get

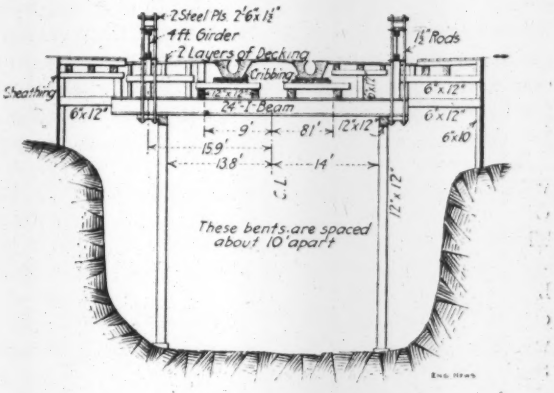


FIG. 41. TIMBERING SUPPORTED FROM LONGITUDINAL GIRDERS ON STREET SURFACE, LEXINGTON AVE., SECTION 10.

an arching effect is, however, to be noted, as this principle was applied also on Section 1.

The features of this system as developed on Section 1, by J. C. Meem, who was the engineer in charge for the contractors, are the combined arching and bracing effect secured and the use of the continuous girder throughout the length of the excavated section, supported on posts or hollow piles.

At a convenient distance below the decking, continuous 12x12-in. cross-braces spanning the whole width of the excavation were put in, 10 ft. apart. These were usually made up of two 6x12-in. timbers bolted together in as long lengths as it was practical to use. The short 12x12-in. cap, with the ends cut as shown, was then placed under the cross-braces and supported temporarily while the next lift of 5 or 8 ft. was excavated to the level of the next cross-brace.

The hollow-steel piles were then driven, a 10x10-in. post fitted inside of them, the longitudinal girders extended, and the arch legs and posts put in position. The appearance of the lowest level is shown in Fig. 37.

The use of continuous I-beams for the support of the street decking or for tying together the timbering or supports is a feature of quite a number of the sections, differences in detail, of course, having been developed by each individual contractor, but in general the principle is that of a group or series of groups of I-beams spliced together with long plates to obtain 100% strength at the joints and running the whole length of the work under the decking and often supporting directly the street-car tracks. The advantage of this method, of course, is in its general security in case of accident to any one or even more of the supporting columns or bents and also to permit easy removal or change of supports during erection.

An interesting and unusual example of the method of timbering by rangers and braces applied to the excavation of a large area in water-bearing material is that of Sec. 2-a at the intersection of Broadway and Canal St., which, on account of its difficulty, was let as a separate contract. A double-deck structure is being provided to permit the future Canal St. cross-town line to pass under the Broadway line. The excavation at this point is some 55 to 60 ft. deep and for four-track lines approximately 250 ft. long on Broadway and 150 ft. on Canal St., joined by a connecting curve in the northeast corner. The extremely heavy floor of steel girders and concrete placed in the bottom of this section to resist the water pressure, as shown in Fig. 11, will convey an idea of the conditions which were successfully overcome.

The normal water level on this section is about 15 ft. below the surface of the street, but water was actually not found in quantity until a depth of about 23 ft. was reached. This left, however, a depth of 30 ft. to be excavated in water-bearing sand and gravel. The entire area is surrounded by buildings and there is a very heavy travel to support on both streets, so that it was necessary to exercise extreme care to avoid losing ground.

The area excavated is surrounded by 6-in. tongued-and-grooved sheeting driven in three lifts, and great care was necessary in spacing the elaborate system of cross-bracing to permit the erection of the steel. There is nothing particularly novel in the layout of this latter, except its great extent over such a large area; special care, of

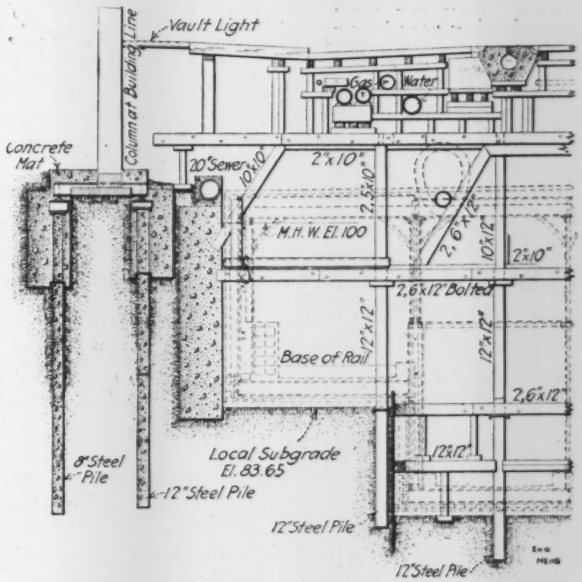


FIG. 42. CONCRETE SIDEWALLS AND ARCHED TIMBER SUPPORTS WITH HOLLOW STEEL PILES USED ON BROADWAY, SECTION 3, ROUTE 5

course, being necessary to keep the long line of rangers and braces in place with proper cross-bracing, and to so design the whole layout that the steel could be erected with the minimum amount of interference with the timbering.

Fortunately, the large amount of water coming into the excavation seems to be quite clear, and the most careful levels, carried out for a considerable distance in every direction from the excavation and continually checked, indicate little, if any, settlement of the adjacent ground or buildings. The total capacity of the pumps is about 20 million gallons daily, and while the whole capacity has only been infrequently required, a large proportion of it has been needed continuously.

City Switching Service with Geared Locomotives

The switching at industry tracks is one of the troublesome features in railway terminal service, due to the limited space available for sidings and connections. This necessitates the use of very sharp curves, which may be combined with steep grades in cases where the industry tracks are above or below the main-track level. The conditions are specially severe in the comparatively few cases where industry service is given to the business districts of large cities. The Kansas City Southern Ry. meets conditions of this kind at Kansas City, Mo., where its industry connections include depressed tracks leading into the basements of large establishments, and for this service it is using geared locomotives.

This industry service is an important feature both to the railways and the shippers, and the Kansas City Southern Ry. has made special endeavor to give such service to warehouses and wholesale houses, etc., within the business district. The fact that the railway can deliver cars at these establishments is an aid to it in securing traffic, while for the shipper it means the advantage of eliminating all team service to and from the freight houses and team yards. The section of the city thus served is accessible from the freight terminals only by streets having very heavy grades (about 7% maximum), which involved very heavy expense for teaming.

There is a considerable amount of industrial work on grades varying from 0.5 to 6.6 and even 7%, and on curves of 16° to 48°. The operation of switching service under such conditions is apt to be dangerous, particularly when the tracks are greasy or covered with ice or snow. In fact the ordinary engines sometimes could not be operated on the maximum grades, but with the geared locomotives there is no trouble in operating in any kind of weather. A special advantage of these engines is the absence of the noisy exhaust of ordinary engines operating on steep grades, which noise (and the accompanying discharge of cinders and sparks) would be highly objectionable in a city district.

The worst conditions are a combination of 7% grade with a 48° reverse curve having a tangent 34 ft. long. Other 7% grades have 60° curves (not compensated). The length of maximum grade is 1050 ft. Under these conditions, the ordinary switching locomotives of 34,000 lb. tractive power could handle only two cars, and similar engines having sufficient power to handle six or eight cars would be too large and too long to be operated under such track conditions. Two geared locomotives were purchased for this industrial service, but are now used also in general switching work. Their transfer work covers a distance of seven miles, and the industry work covers a territory of about ten blocks. They can push or pull trains of four to eight cars on the severe grades and can control them without the use of the brakes on the cars.

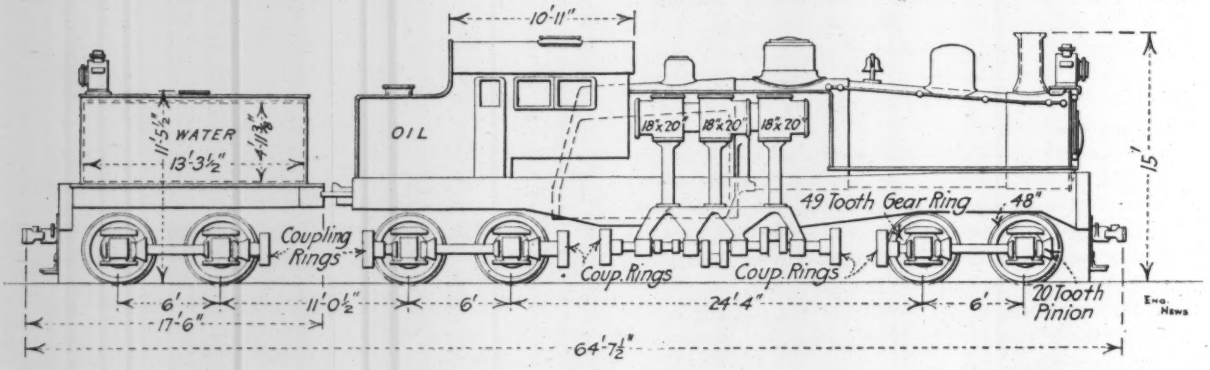


FIG. 1. 190-TON GEARED LOCOMOTIVE FOR SWITCHING SERVICE ON 7% GRADES AND 60° CURVES; KANSAS CITY SOUTHERN RY.

From the railway company's terminal a main industry track leads up a steep grade and along the side of a street in the warehouse district and has branches laid along alleys. From the main-track lead sidings which occupy sidewalk space and run into the basement levels of the building (the siding being parallel with the main track but on a descending instead of ascending grade). From the alley lines, spurs lead into the various establishments, no cars being allowed to stand in the alleys.

In this wholesale district, no objection is made to the occasional interruption of sidewalks. The railway company paves and maintains the paving along its tracks. Turnouts are placed where needed and where they are likely to be needed in future, so as to avoid tearing up paving and tracks. These turnouts have their rails riveted to steel ties (of 8-in. channels) embedded in concrete, thus making a substantial and permanent construction.

Owing to the way in which these engines run in and out of the "holes," they are known among the switchmen as "alley rats."

The engines are of the Shay type, having a vertical inverted engine at one side of the firebox, with its crankshaft connected by intermediate shafts to a shaft on each truck. These last shafts carry bevel pinions (20 teeth) engaging with bevel gears (49 teeth) on the faces of the wheels. Each engine has three four-wheel trucks; one under the boiler, one under the cab and the rear bunker (for oil fuel), and the third under an independent (but power-driven) tender or water tank.

The larger engine, shown in Fig. 1, weighs (with driving tender) about 190 tons in working order, has 74,400 lb. tractive power and will push or pull a train of 250 tons up a combination of 7% curve and 60° curve (not compensated) at a speed of 4 mi.p.h. The smaller engine, Fig. 2, weighs about 140 tons, has 59,800 lb. tractive

power, and is designed to handle a train load of 170 tons on a 7% grade, and to pass, with its train, a 48° reverse curve with 34-ft. tangent and tracks spaced 13 ft. c. to c.

The leading dimensions of the larger engine are given in the accompanying table. The locomotive is practically a double-truck machine with power-driven tender, and the boiler and rear bunker are carried by two heavy fish-belly frames. The engine being at one side of the locomotive, the center line of the boiler is offset from that of the track. On the right side (carrying the engine) the frame is 2 ft. 9½ in. from the center of track, while on the left the distance is 5 ft. 3½ in. The engines were built by the Lima Locomotive Co., of Lima, Ohio. For information we are indebted to C. E Oakes, Mechanical Engineer of the Kansas City Southern Ry.; A. N. Reece, Office Engineer, and C. W. Streeter, Superintendent of Terminals.

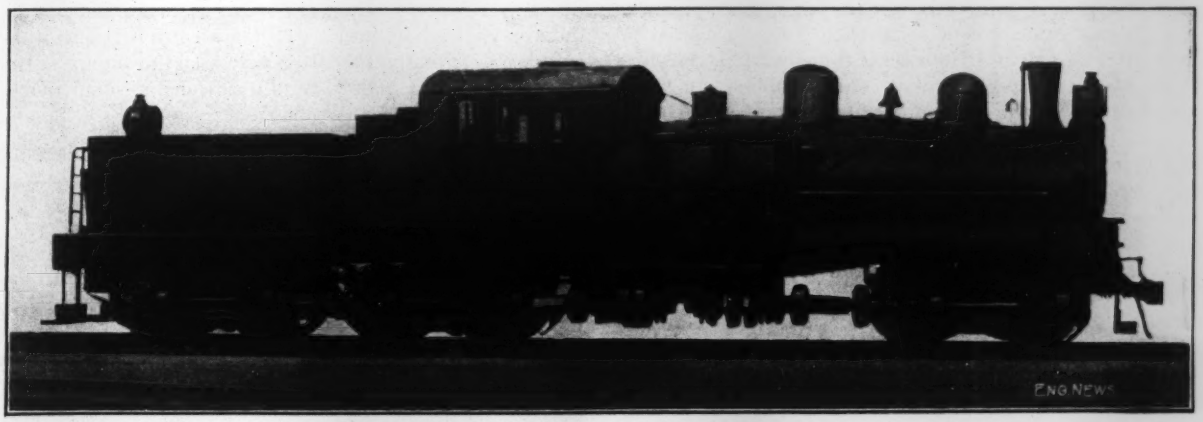


FIG. 2. 140-TON GEARED LOCOMOTIVE FOR SWITCHING ON 7% GRADES AND 60° CURVES; KANSAS CITY SOUTHERN RY.

190-TON GEARED LOCOMOTIVE FOR SWITCHING SERVICE —KANSAS CITY SOUTHERN RY.

Driving wheels (3 trucks), 12	4 ft. 0 in.
Wheelbase, truck (and rigid)	6 ft. 0 in.
Wheelbase, engine	36 ft. 4 in.
Wheelbase, total	53 ft. 4½ in.
Weight on front truck	128,900 lb.
Weight on intermediate truck	138,820 lb.
Weight on tender truck	114,150 lb.
Weight of engine (2 trucks)	267,720 lb.
Weight of engine and tender	381,870 lb.
Weight of engine, light	209,500 lb.
Weight of tender, light	71,700 lb.
Cylinders (3), vertical	18x20 in.
Valve gear	Gooch
Boiler, diameter at first ring	5 ft. 7⅜ in.
Boiler, rail to center line	8 ft. 7 in.
Firebox, size inside	10 ft.x5 ft. 8½ in.
Firebox, height, front	6 ft. 5⅜ in.
Firebox, height, back	5 ft. 11⅜ in.
Tubes, diameter, 2 in.; length	16 ft. 0 in.
Heating surface, tubes	2890 sq.ft.
Heating surface, firebox	208 sq.ft.
Heating surface, total	3098 sq.ft.
Grate area	57 sq.ft.
Fuel (oil), in engine bunker	2200 gal.
Water, in tender	5000 gal.
Length over couplers	64 ft. 7½ in.
Width over running boards	10 ft. 10½ in.
Height, rail to top of smokestack	15 ft. 0 in.
Tractive power	74,400 lb.
Gear ratio	49 to 20

Limitations of Water Filters*

By GEORGE W. FULLER†

While for some years the question of the proper loading or burden on filters has been considered in many places and attempts have been made to fix some sort of a standard which would serve the purpose, these attempts have met with very little success. The difficulties of fixing such a standard are self-evident. It is rather obvious that no exact relationship exists between the sanitary quality of the water and the number of bacteria present, no matter how the bacteria are tested, whether in the form of total bacteria or some special form, such as the colon bacillus. It is possible that some waters should have good sanitary properties and yet have a rather large number of bacteria. Other waters, again, which might be relatively free from bacteria, would show themselves to be unhealthful.

While it has been clearly recognized that some limits, definite if possible, should be placed on the proper burden for filter plants, no such limit has, until recent times, been established.

INTERNATIONAL JOINT COMMISSION INVESTIGATION

An International Joint Commission, composed in part of appointees of the United States and in part of appointees of Canada, has been investigating the question of pollution of the international boundary waters, with the idea of determining to what extent pollution must be limited in order not to endanger the health of the communities on the two sides of these boundary waters. The question practically resolves itself to this:

1. Is it possible to maintain the water in such shape that without treatment it shall be suitable for drinking water?

2. If it is not possible to maintain the water in such a condition of purity and it does need filtration before being suitable for drinking water, what is the limit of pollution allowable before the filtration plant will be overloaded and safe drinking water cannot reasonably be obtained by ordinary filtration?

The first source of information is those cities which have been receiving a supply of unfiltered water and which have sufficient data available to show how this water stands in rank on the basis of B. coli present in the water. A second source of information is the various filtered waters which are supplied to a number of communities and the records they have of coli content in this filtered water and the B. coli content in the raw water supplied to those filters. The most of our communities which do have filtered water show a satisfactory effluent and a satisfac-

*Extracts from a paper before the American Society of Municipal Improvements, Boston, Mass., October, 1914.

†Consulting Engineer, 170 Broadway, New York City.

tory typhoid-fever rate, which is some measure of the wholesomeness of the water supplied. An examination of the corresponding raw waters applied to the filters gives some indication of where a reasonable standard limit should be placed. A third source of information is the consideration of a standard of efficiency of bacterial removal and the assumption of a reasonable number of B. coli in water supplied for drinking purposes and a corresponding calculation of what would be a proper natural limit of B. coli in the water supplied to the filters under these conditions.

The best that can be done with the data at hand is to consider and properly weigh them, and on the basis of a rather general judgment come to some conclusion on what is a proper limit of B. coli in water to be applied to the filters. Such a conclusion should be considered an average rather than a rigid limit for any particular case. This standard specifies that water applied to filters should show by the presumptive test not more than 500 B. coli per 100 c.c. of water as a yearly average, meaning by this that B. coli should not be found more than 50% of the time in 0.10-c.c. samples. For averages for shorter selected times, monthly, weekly, or daily, the allowable B. coli content may be considerably higher. It is believed that for this purpose the averages based on a year's readings are more useful than averages for a shorter time with a corresponding different standard of allowable B. coli.

The bacterial efficiency of a filter is not an invariable quantity. Roughly speaking, 98 to 99% may be placed as a fair average under ordinary conditions. Percentages, however, are not always a good gage of efficiency, as a water with an initial bacterial content that is high will show a proportionately higher efficiency in a filter than a water with a low bacterial content, and yet the resultant filtered water may be much less satisfactory to the consumer from a health standpoint than would be the case with the other filter with a lower bacterial efficiency. In addition, sterilization is usually applied in modern filter plants as a reserve factor of safety, and should be available to be applied in all plants, and with the use of effective sterilization added to properly filtered water an effluent can be obtained which is almost, if not entirely, sterile for ordinary use.

With these conditions in mind, it seems reasonably safe to say that a water having in its raw form a coli content not exceeding 500 per each 100 c.c., based on yearly averages, will show in the water supplied to the consumers something not more than five to ten B. coli per 100 c.c. Such water is believed to be a fairly safe water, when properly sterilized, for drinking purposes and for all other domestic uses.

A Small Reinforced-Concrete Dam with a Large Upstream Apron of Concrete Paving

(*Contributed*)

A private reservoir recently created in northeastern New York has become locally famous because of the large amount of concrete placed for such a modest project. Inquiries and comments show that a certain knowledge of the peculiarities of the construction is spreading so that all further available details should be recorded. The project will be recognized at once by those who have previous knowledge of the work; to others it will be the details and not the persons involved that are of interest, so that names have not here been given. The reservoir in question is for purely private service and exact figures on the design and construction have been refused; however, such as are given below are believed to be closely approximate.

In 1909, work was started on a small dam to impound the waters of a little stream, flowing toward Lake Champlain, so that the seasonal flow might be equalized for the better development of electric power on a large farm where many operations were carried on by motor-driven machinery. Surveys were made by a firm of civil engineers in a near-by city and designs were worked out by them in connection with a well known firm of consulting engineers. Finally the latter were asked to supervise the construction of a hollow reinforced-concrete dam in accordance with the plans submitted.

These plans called for a deck, supported by triangular

FIG. 1. DOWNSTREAM SIDE OF DAM, SHOWING GATE HOUSE AND UNASSEMBLED STEEL CONDUIT

buttresses, approximately 1700 ft. in length, with its crest 37 ft. above the stream bed. The initial contract was placed with a local contractor and the work of clearing the reservoir site of undesirable material was immediately started. The cleared area totaled 160 acres, the greater part of which was solid rock floor.

Little trouble was experienced during construction, until a portion of the structure, located east of the stream, was reached. But there, interesting conditions were met with in the excavation of the foundation trench. While along that portion of the dam west of the stream and extending to its east bank the rock appeared and formed the natural surface, immediately upon leaving this east bank the drift of the rock away from the surface was so very rapid that it was found necessary to dig a trench 40 ft. deep, more or less, and at some points nearly as wide, in order to obtain a satisfactory foundation for the footings.

No little difficulty was experienced in making this excavation, owing to the unusual number of large boulders, distributed at various depths throughout the entire length of this portion of the trench. These varied from 1 to 5 ft. in diameter and were surrounded by coarse gravel, much of which was so compact that its removal

was made possible only by the use of dynamite. The hazardous nature of the excavation and the trouble experienced in retaining sheeting in place after driving made it almost impossible to keep laborers on the work for any length of time.

A 100-ft. spillway, of the ogee section, is located at the western end of the main dam, at an elevation 2 ft. lower than the crest. It is equipped with adjustable flashboards, constructed from 3-in. plank, securely cleated together and attached with strap hinges to a 6x8-in. sill that is firmly anchored to the spillway top. The boards are braced against the downstream toe of the dam.

From the western end of the spillway, a cutoff wall of gravity section, having a batter of 6 in. to the foot, upstream, was extended for 325 ft., more or less. Likewise from the eastern end of the main dam a wall of similar design, but of greater dimensions, was extended several hundred feet into the slope.

After the structure was completed according to plans, an attempt was made to fill the reservoir, with decidedly unsatisfactory results. Owing to the porous character of the soil in the east slope, the water upon raising its level began a steady seepage, which increased rapidly with the increased head, to the end that one farmer, living some distance outside the eastern slope, was seriously inconvenienced by the flow. This result was not wholly unexpected, as before construction had started the consulting engineers must have pointed out the very questionable nature of the site on account of the character of the east bank. Indeed the whole countryside is known to have been subjected to severe glacial action. The owner, however, very greatly desired the reservoir at this point and evidently ordered the work to proceed in spite of the undesirable conditions.

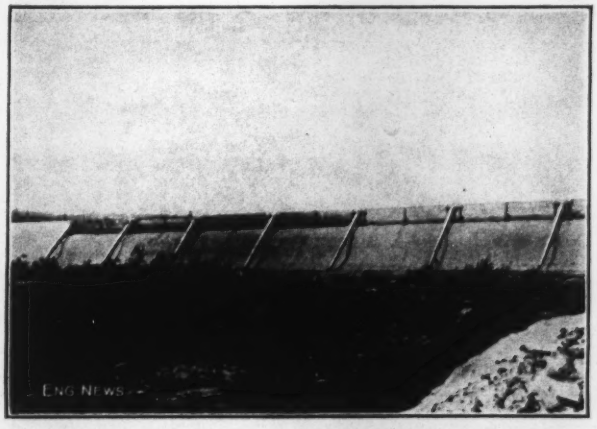


FIG. 2. SPILLWAY WITH ADJUSTABLE FLASHBOARDS

At this time any immediate attempt at storage was abandoned, and it was decided by the engineers to lay a sheet of concrete, 6 in. in depth, more or less, over that portion of the south slope—some 50 acres or more in extent—through which the leakage occurred. As an additional preventive, a cutoff wall was extended in a northerly direction, approximately, paralleling the stream for a distance of 2500 ft., more or less, this wall forming an impervious joint with the floor sheet and going down to rock. It reached a depth of 35 to 40 ft. for a considerable distance.

The heavy seepage was stopped, but in the summer of 1914 there was still considerable water leaking out. There may be a seam in the rock or places where the cutoff wall does not connect with rock. One reason, undoubtedly, lies in the large and numerous cracks in the concrete floor due to temperature changes, settlement, shrinkage, etc. There are also cracks at the junction of floor and cutoff wall. Very recently, the reservoir has been drained, the floor washed down and the cracks repaired; probably, the necessity for such repairs was

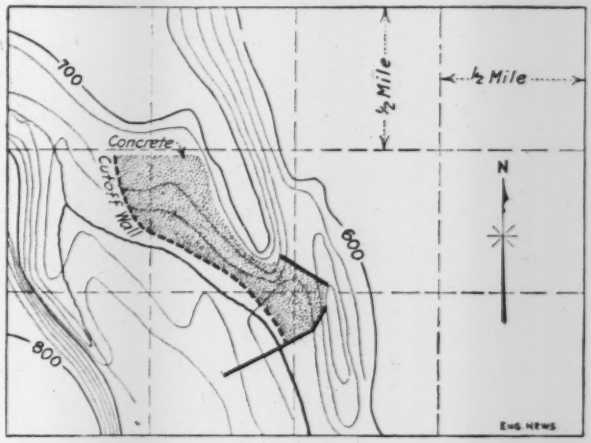


FIG. 3. SKETCH SHOWING APPROXIMATE LOCATION OF DAM AND CONCRETE APRON

fully known before the concrete was laid and no doubt they were judged to be preferable to greater attention at first to expansion joints, better foundation, etc.

The work on the penstock and power house of this project is progressing rapidly and there is little doubt as to the final technical success of the undertaking. It is obvious, however, that the expense incurred in the whole work was far beyond what would be possible for any commercial enterprise in a hydraulic development of such size and nature. The first estimates of the cost of the project were in the vicinity of $200,000. It is reported locally that the whole work, when completed, will approach $1,000,000.

The Largest Chain-Drive Transmission in the World is said to be that of the Ox-Bow hydro-electric plant on the Snake River at Copperfield, Ore. The installation consists of one 3600-kw., three-phase, 60-cycle generator, running at 225 r.p.m., operated by two water-wheel units. Each of these units consists of two pairs of water wheels of the horizontal type, with 48-in. center discharge, set in open flume, and operating at 21 ft. head and 147 r.p.m. The water is supplied through a tunnel cutting off a bend in the river. Each water-wheel unit is connected to the generator by four Morse chains 21 in. wide, with sprockets of 2-in. pitch on the line shafting, and with shaft centers 10 ft. apart. The generator shaft is in the center and 5 ft. above the level of the water-wheel shafts. This chain drive is said to be three times greater in size than any previously built, the largest one previous to this being at Indianapolis, Ind., and driving a 1200-hp. generator. The Ox-Bow plant was designed originally to operate under a 45-ft. head obtained by a natural fall of 21 ft. by the tunnel through the bend in Snake River, and the building of a diversion dam across the river below the tunnel. Both the water-power and electrical machinery were purchased for this head, but due to financial troubles the company was unable to build the dam and for four years the property lay idle. The plant was put in operation on Sept. 30. It was designed by the W. H. Rosecrans Engineering Co., of Chicago.

Field and Office

Quarrying with Air Hand Drills on Scaffolds, Slings and Ladders

By Charles C. Phelps*

Certain new methods of ledge and cliff excavation have been successfully developed since the advent of the self-rotating hammer hand drill which would have been impractical with all previous types of drills. Formerly, the familiar bench method was usual, the rock being removed in steps whose depth and width depended upon the size of mounted drills employed. Frequently deep holes were

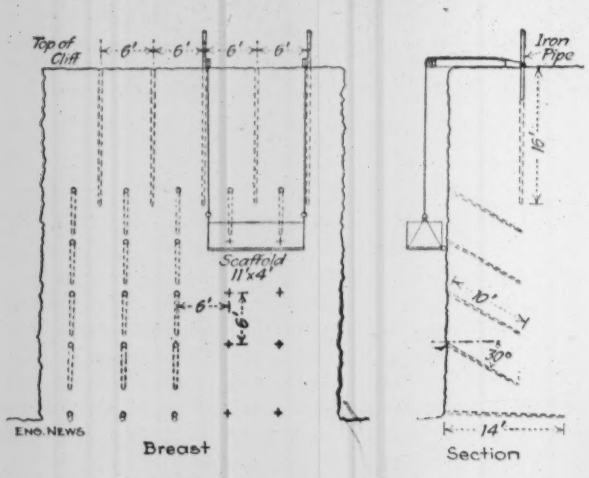


Fig. 1. Sketch of Breast Drilling at Campbell & Deane Quarry, Straw Plains, Tenn.

cut from the top of the cliff by churn drills and drill wagons.

In the new method, a support for the workmen is lowered over the edge of the cliff. There being no drill mounting to adjust, the time consumed in setting up and in changing steels is practically eliminated. This always consumes a large part of the time of operation with all types of mounted drills and explains the great economy of hand drilling. Yet the methods here described would have been impractical with earlier hand drills, due to the difficulty of rotating them by hand in inconvenient positions.

Fig. 1 and 2 show the scheme employed at the Campbell & Deane Co. quarry, Straw Plains, Tenn. The rock is a hard blue limestone. Formerly, mounted piston drills (3½x6-in. cylinders) were employed to remove the rock in benches. These were capable of drilling holes up to about 2¾ in. starting diameter, and a maximum depth of about 16 ft.

The present procedure is as follows: A row of vertical holes, 16 ft. deep and spaced 6 ft. apart, is put in parallel with the face of the cliff by the tripod drills. Two lengths of gas pipe are inserted in the proper holes and ropes are fastened to them so that a scaffold can be lowered over the face of the cliff. The scaffold, shown in Fig. 2, is 11 ft. long and about 4 ft. wide. It is fitted with winding drums so that the drillers can raise or lower it after it has been slung over the cliff. The hand-drill holes are spaced 6 ft. apart, vertically and horizontally, and are 10 ft. deep. At this depth, the steels cut to about 1⅜-in. bottom diameter. The holes are inclined downward at an angle of about 30° from the horizontal. Under present conditions, one man with a hand drill sinks about 80 to 100 ft. of hole per 10-hr. shift.

After the 10-ft. holes have been drilled from the top of the cliff to near the bottom, tripod drills are again put into use for drilling a line of horizontal holes on the very bottom of the cliff, these holes being 14 ft. in depth and looking down slightly from the horizontal. The holes in the side of the cliff are loaded each with 12 sticks of 60% dynamite and those at the bottom with 18 sticks. All of the holes are shot simultaneously (by battery). The cost of powder per cubic yard of broken stone is about 3½c. with this system whereas last year, when only tripod drills were used, the cost was 5½c.

Several big blasts have already been made. In the first shot that was set off an insufficient quantity of powder

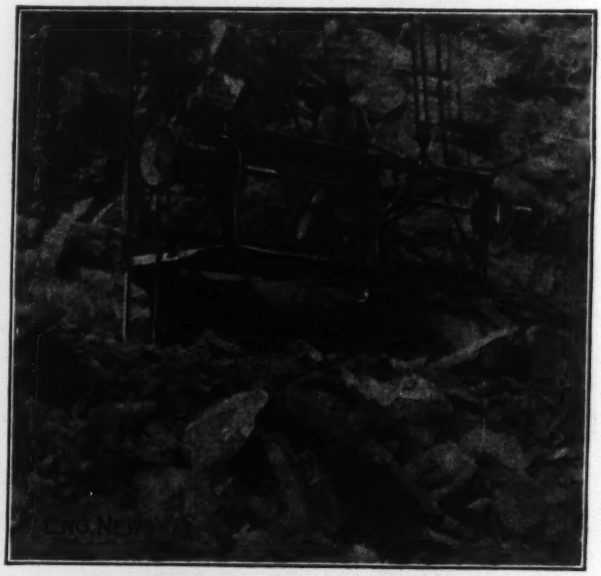
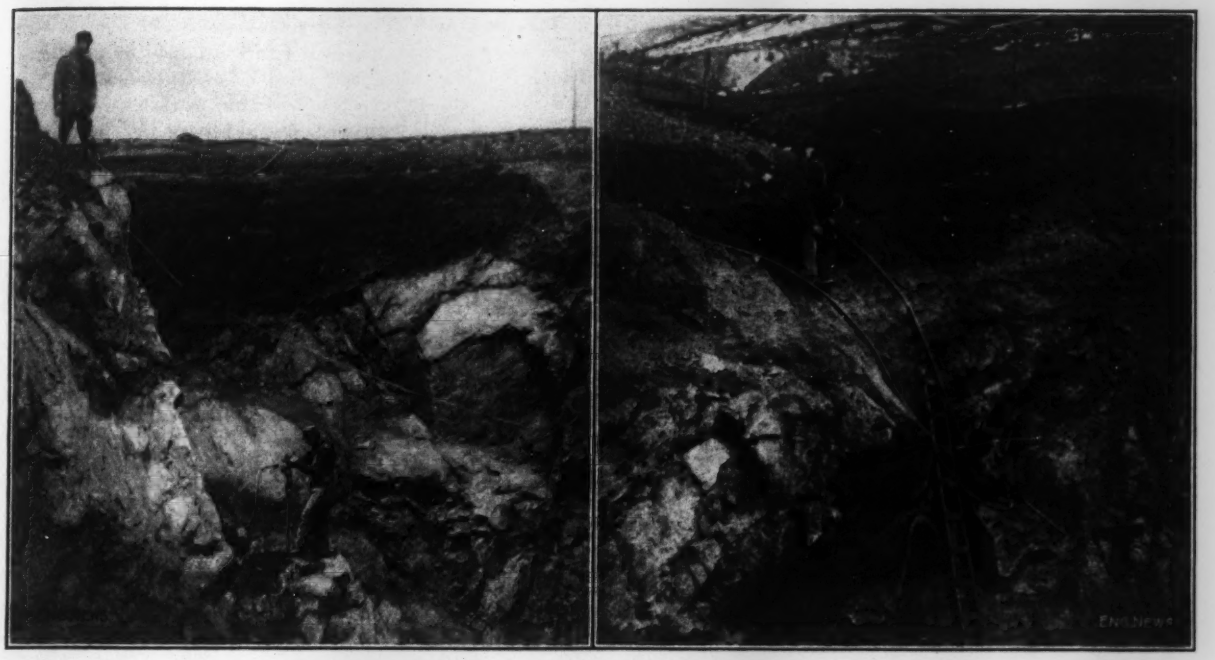


Fig. 2. Scaffold Near Base of Cliff, Campbell & Deane Quarry

was used, which resulted in large sizes of some of the stone. Later, 350 lb. was used (as noted above), and the cost quoted is based on this. It is believed that this amount can be reduced slightly and it is also contemplated using a slightly longer scaffold so that two or three drills can be used at once, which would result in a further low-

*Ingersoll-Rand Co., 11 Broadway, New York City.



FIGS. 3 AND 4. HAND DRILLS IN USE AT BULLWHACKER MINE, BUTTE, MONT.

ering of operating cost. With the 350-lb. charge of powder, the rock is so well broken up that practically all of it can be handled by the steam shovel and crushers, requiring practically no block-holing.

The ease with which the automatically rotated hand hammer drill is handled in inaccessible quarters is further illustrated by Fig. 3, showing work at the Bullwhacker Copper Co. mines in Butte, Mont. The difficult nature of the "glory-hole" mining here may be realized when it is considered that it is 75 ft. from the operator's position to the bottom of the cut, where the broken ore is mucked into cars and hoisted on an incline to the surface. The material is relatively low grade, averaging 4% copper, the vein running about 125 ft. wide at this point.

Frequently the operator works from a boatswain's chair slung over the side of the cliff. Fig. 4 shows a scheme for drilling in a particularly difficult spot where a suspended ladder had to be resorted to. The operator is braced against the ladder and has only sufficient room for one foot on the rock. In spite of the exceptional difficulties at the Bullwhacker mine, the remarkable speed of 90 ft. of hole in two hours has been made, each hole bottoming at 6 ft. The rock, however, was not hard, it being an oxidized granite. Three-foot starting steels were used, followed by 6-ft. drills, the holes being for 1¼-in. powder sticks. In a single month, one drill broke enough ore to fill 64 cars of about 50,000 lb. each.

In the two instances cited, it would be quite impossible to employ heavy piston drills. Ordinary plug drills would also be extremely difficult to operate because of the difficulty of rotating them by hand in such cramped quarters. With the self-rotating hand type, however, the operator has merely to apply the tool to the spot to be drilled and open the throttle. A simple steel holder attached to the front head of the hand drill has been found of material aid in withdrawing the steel from the hole. Moreover, it enables the drill runner to lower and raise the steel in the hole, churning up the cuttings and preventing sticking or binding. The latter troubles are still further provided against and the speed of cutting has been increased by a hole-cleaning device with which the operator can send at will a jet of air through the hollow steel to the bottom of the hole.

The writer desires to acknowledge the assistance of L. F. Thompson, of Knoxville, who furnished the data concerning the Campbell & Deane quarry, and of C. M. Hansen, of Butte, who supplied the information on the work at the Bullwhacker mine.

An Overturned Retaining Wall—That structures do not always do what is expected of them is illustrated by the adjoining picture of a 4-ft. retaining wall on Panorama Way, Berkeley, Calif. We do not know whether the wall was designed or built by eye, but it had the strength of a revetment rather than of a retaining wall, though carrying a surcharged

RETAINING-WALL OVERTURNED BY SLIP OF CLAY BACKING, BERKELEY, CALIF.

clay bank. It was only 10 in. thick, without batter front or back, reinforced in the main horizontally (i.e., not at all as regards retaining-wall action), and provided with only occasional counterforts. Worst of all, the soil here is very liable to slip during the winter rains. As a result, the wall recently tipped over as the picture shows. It has been replaced with a somewhat lower wall, of wedge section and counterforted.

Conveying Concreting Materials by Belts, at the Hell Gate Bridge

Belt conveyors have been installed at Randall's and Ward's Island, New York City, for handling concrete material from scows to storage piles, in the construction of the Hell Gate Bridge, crossing the East River. The belt at Ward's Island is 380 ft. long by 24 in. wide and inclined at 21°. At Randall's Island a 20-in. belt is used.

The sand and stone or gravel are lifted by the belt, above storage piles, where they are dropped by a tripper. From the piles they are later rehandled into cars for distributing to different parts of the work. Coal for the dinkey engines is conveyed by the belt in the same manner. The bags of cement are discharged from the belt on a table in a storage house, where they are piled.

The number of men employed on this work at Ward's Island is as follows: A gasoline-hoist operator on the scow; a man to direct the sling; three to make up the slings; four to place the bags on the belt; one to operate the tripper, and 12 to 15 men in the cement-storage room.

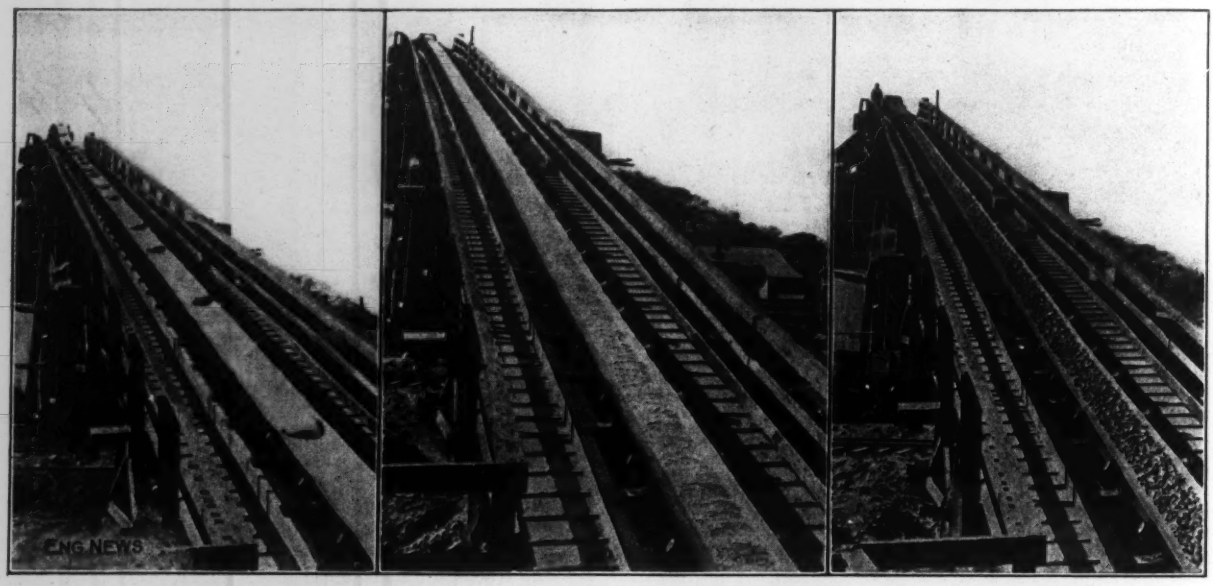


FIG. 1. BELT CONVEYOR AT HELL GATE BRIDGE, HANDLING CEMENT, SAND AND STONE

The receiving dock for materials at Ward's Island is double-decked. The loose materials are unloaded from the scow into a common hopper on the upper deck of the dock, by a derrick equipped with a grab bucket, located on the dock. The belt is fed by a chute from the hopper. The bags of cement are lifted from the scow in 9-bag slings, by means of a gasoline hoist on the scow. Four men, working two and two, unfasten the slings and place the individual bags on the belt. As many as 1500 bags per hour have been handled in this manner without confusion.

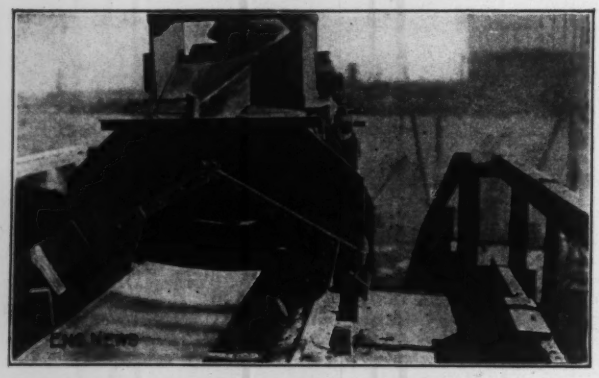


FIG. 2. TRIPPER AT TOP OF CONVEYOR

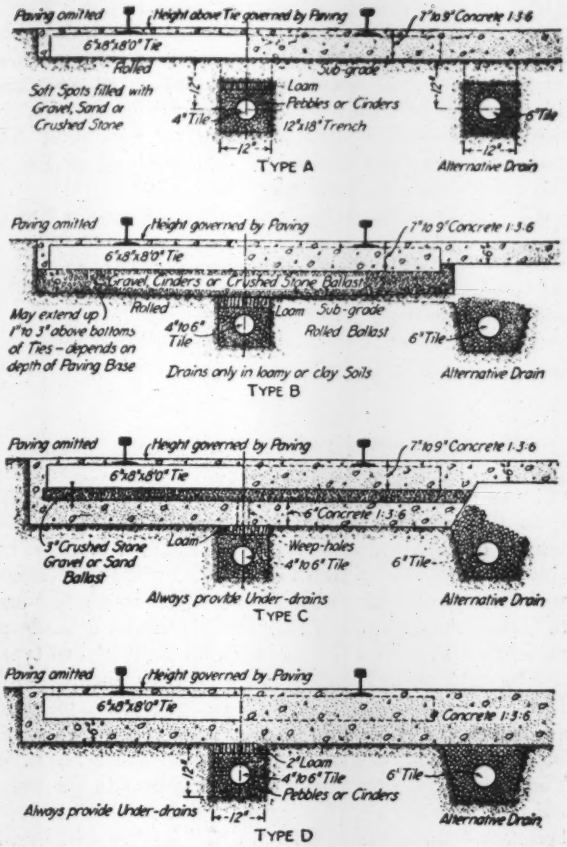
At Randall's Island the scheme of handling is essentially the same, but instead of having a separate hoist for the cement, the one derrick on the dock unloads all materials. To handle cement out of the scow, the grab bucket is lowered into the scow and about 12 bags placed in it and lifted to the dock. At this plant, also, a second belt, 18 in wide, runs from the storage piles to the concrete mixer.

The conveyors were furnished to the Patrick Ryan Construction Co., New York City, by the Robins Conveying Belt Co.

Track Foundations in Paved Streets

The four types of street-railway track foundation shown in the drawings herewith were recommended by the Committee on Way Matters of the American Electric Railway Engineering Association at its recent convention. It is reported that these designs were evolved after study of all influencing factors.

Type A is for moderate traffic of cars up to 25 tons, and for gravel, sand or sandy loam soils. Type B is for most soils except very heavy clays, and for heavy traffic



FOUR TYPES OF TRACK FOUNDATION IN PAVED STREETS

(Recommended by the Committee on Way Matters of the American Electric Railway Engineering Association.)

of cars up to 35 tons. Types C and D are for heavy water-retaining soils and uncertain made ground, with heaviest traffic and cars. Surface drains are to be provided in all cases.

Some Neglected Points in the Theory of Adjustment of the Engineer's Transit

BY J. A. KITTS*

The writer outlined in the ENGINEERING NEWS, of July 9, 1914, a method of using and adjusting the wye-level, based on the operation of maintaining the vertical axis of the instrument precisely vertical. In using the transit, it is also essential for successful operation that the vertical axis be kept vertical—operation (1) following)—as the horizontal circle needs be horizontal to measure horizontal angles. The vertical axis is necessarily made perpendicular to the plane of the horizontal circle by the instrument maker.

It being shown by application of operation (1) that the vertical axis can be made vertical with fine precision and quite simply even though the plate or telescope bubble tubes are out of adjustment, the adjustment of all the parts is greatly simplified and facilitated and, as far as possible, made independent of each other. The precision of this operation, as also that of any method of adjustment, depends on the spindles and bearings fitting properly, being circular in section, and the outer and inner spindles being concentric. This provides an index to a defect in the spindle.

It will be observed that only one bubble tube is necessary on the complete transit, that on the telescope. One bubble tube on the plate is quite sufficient for convenience in the operation of leveling the instrument and should be placed at right angles to the telescope bubble tube.

In a theoretically perfect transit instrument the following specifications obtain:

(*a*) The inner and outer spindles, horizontal circle and verniers are concentric.

(*b*) The spindles or vertical axes are perpendicular to the plane of the horizontal circle.

(*c*) The horizontal axis is perpendicular to and intersects the vertical axis.

(*d*) The line of sight coincides with the optical axis, is perpendicular to the horizontal axis and intersects the horizontal and vertical axes.

(*e*) The tangential axes of the plate bubbles are perpendicular to the vertical axis.

(*f*) The tangential axis of the tesescope bubble is parallel to the line of sight and optical axis of the telescope.

(*g*) The cross-hairs are horizontal and vertical respectively and intersect on the optical axis.

(*h*) The plane of the vertical circle is perpendicular to the horizontal axis. The vertical circle is concentric with the horizontal axis, and the vernier reads 0° on the vertical circle when the line of sight is horizontal.

(*i*) The circles are graduated uniformly.

Physical changes take place in the instrument, usually on account of rough usage, and it is necessary to test the adjustments often. The following method of adjustment has been found to facilitate a sometimes tedious process.

ADJUSTMENTS

(1) *Make the vertical axis vertical.*

Bring the plate bubbles to the center over both pairs of leveling screws. Set the vertical circle at 0° and bring the telescope over one pair of leveling screws. Note the end of the bubble near the eyepiece. Revolve 180° in azimuth and again note the eyepiece end of the bubble. Bring the end of the bubble to the mean position over both pairs of screws. The bubble will then remain in the same position for any angle about the vertical axis and the vertical axis is vertical. It will then be observed that the plate bubbles remain in the same position for any angle about the vertical axis, and either of these bubbles may be used instead of the telescope bubble. The telescope bubble, however, is the more sensitive.

(2) *Test the inner and outer spindles for eccentricity.*

Make the vertical axis vertical by (1). Hold upper plate and turn lower plate through a complete revolution. If the bubble remains in the same position for any angle of the lower plate in reference to the upper plate, the spindles are concentric. If not concentric, the correction should be made by the instrument maker.

(3) *Adjust the plate bubbles.*

Make the vertical axis vertical by (1). Adjust the plate bubbles to central position. The bubble will then remain in this position for any angle about the vertical axis.

*Until recently with the Isthmian Canal Commission, Panama Canal.

(4) *Make the horizontal axis of the telescope perpendicular to the vertical axis.*

Make the vertical axis vertical. Sight on a well defined elevated point, telescope erect. Depress the telescope and establish a point at about 100 ft. from the instrument. Repeat, telescope inverted, and establish a point beside the one first established. The bisector of the line between these two points is in a vertical plane passing through the elevated point and the vertical axis. Maintain the vertical axis in a vertical position and adjust the horizontal axis until the line of sight remains in this plane.

(5) *Make the horizontal axis perpendicular to the vertical axis by means of the striding level.*

Make the vertical axis vertical. Set the striding level on the collars of the horizontal axis. Note the end of the bubble near the fixed end of the striding level. Reverse ends of the striding level and again note end of bubble at the fixed end. Maintain the vertical axis vertical and adjust the horizontal axis, bringing the bubble to the mean position. The horizontal axis will then be horizontal.

The bubble of the striding level may then be adjusted to a central position, although the accuracy of the horizontal axis adjustment does not depend upon the striding level being in perfect adjustment but depends upon the principle of reversion, it being essential that the vertical axis is kept vertical by operation (1).

(6) *Make line of sight perpendicular to the horizontal axis.*

Make vertical axis vertical. Sight on distant well defined point (telescope erect). Transit telescope and establish foresight. Repeat (telescope inverted). The bisector of the line between the two points established is in a vertical plane through the backsight and the vertical axis. Adjust vertical cross-hair, bringing it into this plane. See that the vertical hair follows a point when the telescope is rotated in altitude.

(7) *Make the horizontal cross-hair pass through the optical center.*

Make the vertical axis vertical. Level the telescope approximately. Establish a point on a vertical surface about 300 ft. distant and another at about 25 ft. distant (telescope erect). Invert telescope and sight on the nearer point and check on the distant point. Adjust the horizontal cross-hair to the mean position. See that the horizontal cross-hair follows a point when rotating in azimuth.

(8) *Adjust verniers of vertical circle.*

Make vertical axis vertical. Set vertical circle at 0° and establish a point at about 300 ft. Set the vertical circle at 180° and establish a point above or below the other, as the case may be. The bisector of the line between these two points establishes a horizontal line through the horizontal axis of the telescope. Sight on this point and bring the vernier to 0° or 180°.

(9) *Make telescope bubble tube parallel to the line of sight.*

Make the adjustment supplementary to (8), using the same point determining the horizontal line of sight, and adjust the bubble to the center of the line.

Accuracy in all adjustments depends upon repetition and judgment in choosing distances.

Bulkheads for Boston Dry Dock

In the preparation for the new dry dock in Boston harbor (ENGINEERING NEWS, Apr. 2, 1914, p. 709), the site was paralleled on north and south by large bulkheads, which are to be joined at their outer ends by a coffer-dam to inclose the dry-dock area. Each bulkhead is made up an earth fill between timber bulkhead walls of two types.

The outer wall, about 7500 ft. long, consists of 45-ft. to 50-ft. oak piles, braced by oak spur piles, to which are bolted 8x10-in. wales, holding 4-in. yellow-pine sheeting, 30 ft. long, driven to grade 14 ft. above M. L. W. The sheeting is square-edged and is spiked to the wales. All pile bolts are 1¼ in. diameter, and wale and stringer bolts are 1 in. in diameter. A detail is shown in Fig. 2.

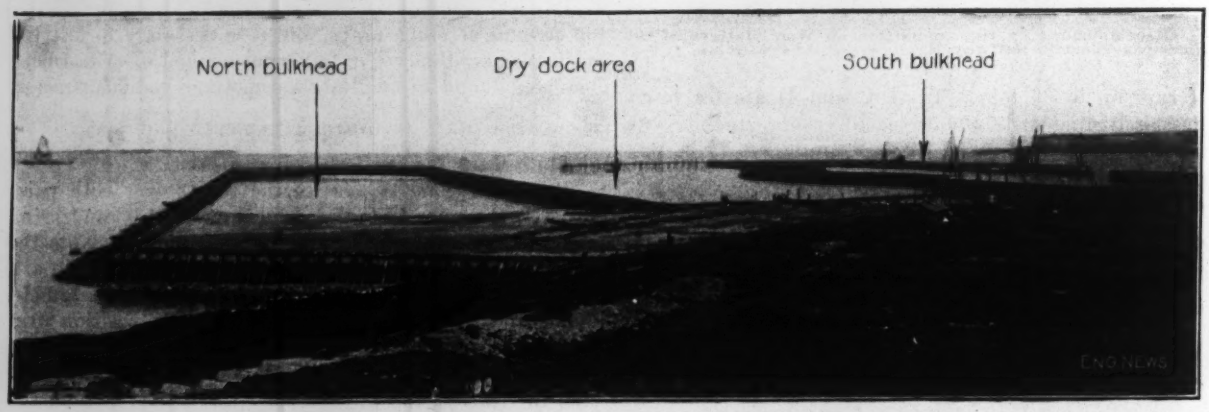


FIG. 1. BULKHEADS AROUND DRYDOCK SITE, BOSTON HARBOR

The inner bulkhead wall, 3200 ft. long, and facing the site of the dock, is constructed of spruce piles of the same dimensions as the outer bulkhead piles, similarly braced and sheathed with 6-in. grooved, yellow-pine sheeting, fitted together with 1½x3-in. spruce splines. No preservative treatment of any kind was used on the piles or sheeting.

A noteworthy feature of both bulkhead walls is the relieving platform of 3-in. oak plank, which is slung from the top stringer of the bulkhead by 2x3-in. oak straps at about the low-water line. The platform is about 4 ft. wide and extends around the entire inside of all the bulkhead walls.

The total area to be inclosed is about 37 acres and

some of the earth fill is to be dredged from the dry-dock site in getting the bottom there down to grade, while the remainder will be dredged from a new channel leading to South Boston wharves.

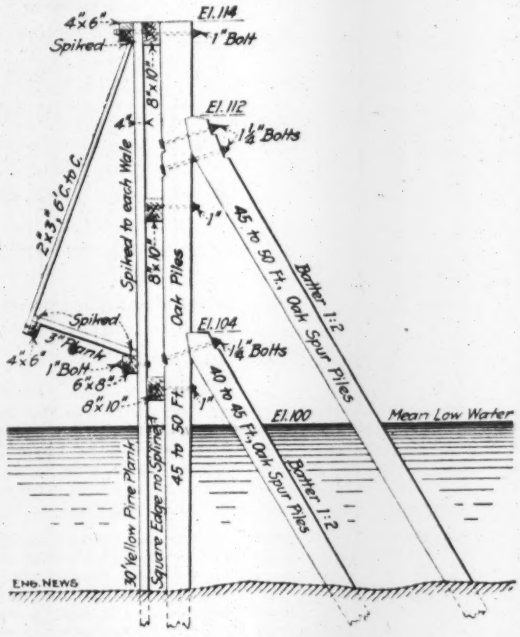


FIG. 2. SECTION THROUGH OUTER BULKHEAD WALL, BOSTON DRY DOCK
(Note relieving platform.)

The work is being carried out by the H. P. Converse Co., of Boston, under the direction of the Directors of the Port of Boston, with Frank W. Hodgdon, Chief Engineer.

Notes from the Experience of an Old Surveyor

BY A. W. BEDELL*

MEASURING LINES—Probably the most reliable method of measuring existing land boundary lines is to set a station within taping distance of every point to be located and measure angles and distances to each of these points, any of which can then be relocated exactly later. Stadia methods will be found as accurate as taping in the case of slight bends in the boundary line, in locating which some error is allowable in the bearing, provided the angle between boundary and traverse lines is small.

There is one disadvantage in this method—from which, however, most of the others are not free—that no matter how good the traverse closure may be, a mistake in one of the side shots will affect the boundary and pass undetected unless all measurements are checked in the field.

Points on a boundary that are inaccessible or difficult to measure to, can often be located by intersection from two stations. This is reliable if carefully checked, but the triangle should be well proportioned with no angle less than 30° or 40°.

ANGLE MEASUREMENTS—There are several ways of measuring angles which are all good so long as consistently used, and the notes should show clearly what is done. Some surveyors measure interior angles only. On a closed figure these should total 180° × (number of angles — 2). Others record the deflections from the line of the backsight produced forward. In this case counting exterior deflections as plus and interior (reëntrant) ones as minus, the algebraic sum for the closed traverse should total 360°.

In all these cases the backsight is set on zero. A common way is to record readings as angles to right or left of backsight or as deflections to right or left of the backsight line produced (Fig. 2).

The bearings of the courses can be figured continuously, checking them with a needle, by considering northeast courses plus (clockwise); left readings will be minus and right ones plus. In a closed figure the algebraic sum of all plus and minus readings should be zero or a multiple of 180°.

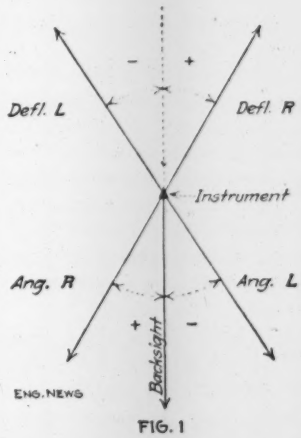


FIG. 1

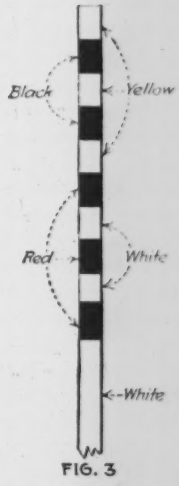


FIG. 3

The most convenient way is to carry the azimuths on the instrument by setting the vernier on the forward azimuth of a line, and backsighting with the telescope plunged; the plate is then always oriented. To do this, the collimation adjustment must be good, or this error accumulates. The advantages of this method are that the azimuth of a line is always known, and that the needle, therefore, gives an instantaneous check. Where greater accuracy is needed angles must be repeated.

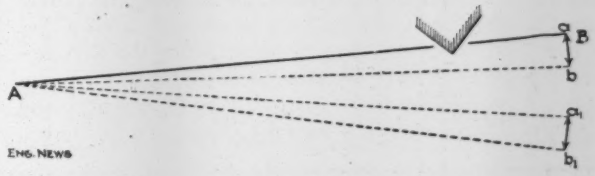


FIG. 2

SIGHTING AROUND OBSTACLES—Very often an obstacle on a line prevents a direct sighting between points. In such cases an equal distance may be offset on each side of the point sighted to, and the angle read at each and bisected. A quick way of getting a backsight or zero when an obstacle intervenes was devised by F. R. Raven, of Glen Cove, N. Y. A stick of any convenient size (a_1 b_1 in Fig. 2) is held somewhere near the backsight point in view of the transit and square with the line of sight AB.

*Chappaqua, N. Y.

The circle is clamped with zero on a_1, the end of the stick nearest to the point, and the upper plate is shifted and set on the other end b_1. The stick is then moved until the end a touches the required point B (invisible); and without disturbing the vernier reading, the transit is oriented on the other end b; whereupon the zero will be in line with the invisible point.

ERRORS FROM DIFFRACTION—There are times when an obstacle is so close to the line of sight that it blurs, although it may not obscure, the object. The writer cautions surveyors against using such sights, for although he has never seen it mentioned elsewhere, it results in errors from diffraction. This is the property of light which causes rays to deflect when passing the edge of an object.

CHECKS—It is a decided advantage to check all measurements when possible; the certainty derived more than compensates for the extra time. Angles may generally be checked by compass readings; stadia sights give a check on distances.

It is best not merely to put check marks in the notes, but to record all readings; they may be useful to refer to if discrepancies arise.

The writer has constructed a range pole with the upper foot subdivided as shown in Fig. 3. Distances are read to all points to the nearest foot at the time of the foresight.

STATION MARKS—Sixty-penny nails as transit points will last longer than soft-wood stakes, and are more convenient to carry and to drive. In fact stakes cannot be used at all in frozen ground. Some surveyors place three stones around each transit point, which makes it easy to locate them even after several years, while a guard stake standing beside the hub might soon be burnt away.

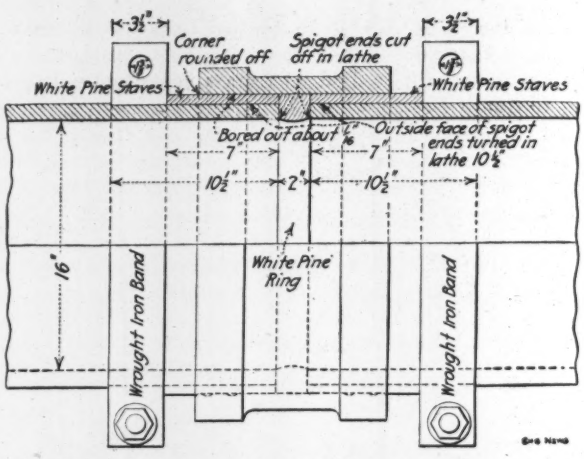
There is nothing that will last longer than stone, and whenever possible it is well to use an outcrop for a station by cutting a deep cross at the survey point.

Important stations on roads, if likely to be disturbed, can be referenced by distances to two or three permanent marks. These may be tacks in trees or fence posts. Only copper tacks should be driven, for nobody uses these except surveyors, while steel tacks might easily be confused with the great number used for bill posting. Moreover, steel will soon rust out, even when galvanized.

When a tack or nail is put in a tree, the spot should first be blazed. This serves to identify the mark, but it may further be of great value in later years by determining the time at which the mark was made. This may be found by cutting out a section of the wood and counting the annual rings of overgrowth. It may not be generally known that a tree trunk does not gain in height after the first few years' growth. Yearly accretion merely adds to the diameter. A nail driven into the wood, therefore, continually becomes covered over, but does not change its position.

NOTES

Wood Insulation Joint for Water Mains—The type of joint shown in the accompanying illustration has been in use in Providence, R. I. (Otis F. Clapp, City Engineer), since 1907. Five 16-in. joints, three 8-in. and fifteen 6-in. joints, as well as several small ones on lead service pipes, are giving good service. The joint is constructed of a regular 16-in. sleeve, bored out $\frac{1}{16}$ in. all around, between which and the pipe are white-pine staves; the thicknesses of these are determined after the machine work on the sleeve and pipe is completed.

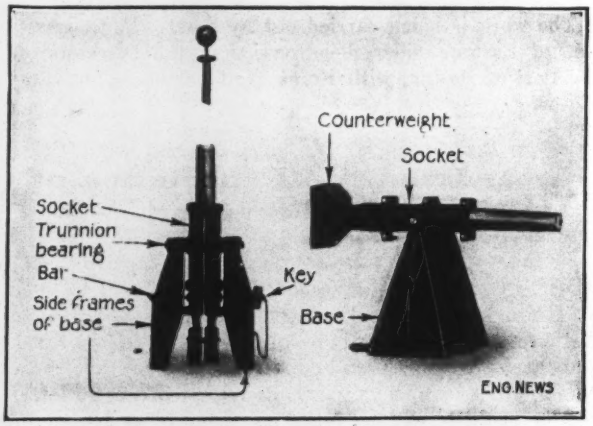


WOOD INSULATION JOINT FOR WATER MAINS, PROVIDENCE, R. I.

Other details are given in the sketch. Very little trouble from electrolysis is experienced in Providence. Where electrolysis has occurred service pipes and meters have been examined and the cause of the trouble removed or the trouble reduced. All the large 16-in. insulation joints are on the high-pressure fire-service mains.

A Tilting Base for Flagpoles—Erecting, inspecting and painting the tall flagpoles commonly used on buildings and in parks, etc., is costly and difficult. The steeplejack is eliminated and the cost reduced by a new type of pole-support in which the flagpole is socketed in a tilting base so that the pole can be lowered to a horizontal position and the painting or rigging attended to without difficulty.

The socket is in two parts, bolted together, so as to firmly clamp the butt of the pole for its entire length. It carries counterweights at its lower end and is mounted by trunnions



TILTING BASE FOR TALL FLAGPOLES
Elmer E. Carr, Chicago, inventor.

in the base, which consists of a pair of castings. The pole is secured in vertical position by a flat bar running through holes in the socket and the base castings, and secured by a pin at the end and a key at the other. To lower the pole, the key is removed and the bar withdrawn, the pole being then readily swung down by hand. It can be secured in horizontal position if desired, to carry a flag in front of the building.

The device is made in several sizes for poles 25 to 50 ft. high, and weighs from 1800 to 2000 lb. For a 40-ft. pole, the side-castings are 5 ft. high, and the socket casting is nearly 6 ft. high, with 3-in. trunnions and three sets of clamp bolts. The counterweights are cast-iron plates of suitable weight, bolted to the flat ends of the socket castings.

This tilting flagpole support is the invention of Elmer E. Carr, 231 Insurance Exchange Building, Chicago. It has been used recently on some large warehouse buildings.

The Protection of Watervliet, N. Y., Against Storm Waters

By F. J. Keis*

SYNOPSIS—The City of Watervliet, to protect itself from periodic inundation by water draining off the rolling land above, has created two reservoirs with outlet works to impound temporarily the storm rainfall, spring freshets, etc., and to allow the floods to pass off at a rate within the capacity of the channel provided. The reservoirs will normally be dry.

The Watervliet Storm-Flood Problem—The City of Watervliet, N. Y., five miles above Albany, is on the west bank of the Hudson directly opposite Troy. Along the westerly limits of the city, the hills rise abruptly some

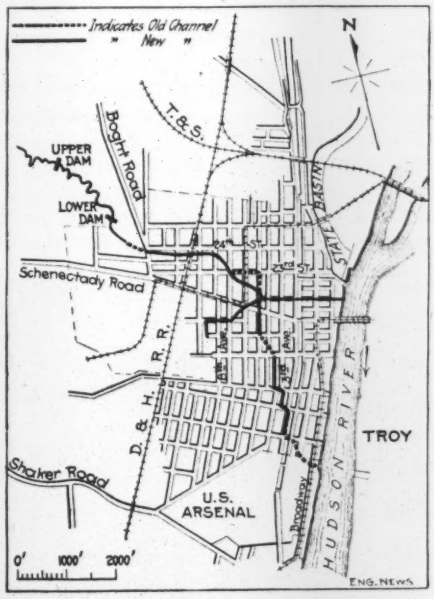


Fig. 1. Watervliet Sketch Map, Showing Dry River and New Storm Sewers

200 ft. and continue back to the ridge dividing the Hudson and Mohawk watersheds. Practically the entire watershed is under a high state of cultivation and is well and rapidly drained. The soil is mostly clay and quite impermeable. The country is open and undulating and the aforesaid conditions, together with the steep gradient of the watercourses, causes the water from melting snow and ice and from even moderate storms to be discharged so rapidly that the capacity of the channels below is overtaxed. The stream which has caused the trouble in Watervliet is the so-called Dry River, and the works herein described have been built for its control.

Dry River entered the northwestern section of the city and flowed through a winding channel in a general southeasterly direction, discharging into the Hudson River at the Watervliet Arsenal. Beyond the city it winds through the hills and for about a mile west of the limits runs through a deep and precipitous gorge which eventually opens out upon the comparatively level city plane, where the stream's gradient is light.

Many culverts had been built to carry the city's streets across the stream, and its waterway had become more or less obstructed by structures arranged to meet individual requirements. In the summer the stream became practically dry except in pools, which served as catch-alls for refuse, garbage and sewage. The filth accumulated between storm intervals was deposited over the streets and in the cellars during the frequent freshets which occurred often with little warning. During the winter months when heavy ice formed, the culverts would clog, force the water over the streets and add to the already intolerable conditions.

A tributary of Dry River, known as "Gas-House Creek," emptied into it about a half-mile from its mouth. Like the main stream, it was an eyesore, a source of constant expense and annoyance, and a danger to the public health.

Various schemes for improvement had been discussed and some had been tried, but these were local in nature and limited in their scope. They generally succeeded in bettering one place only at the detriment of another. In 1910, after considerable agitation on the part of certain civic organizations, it was determined to devise a

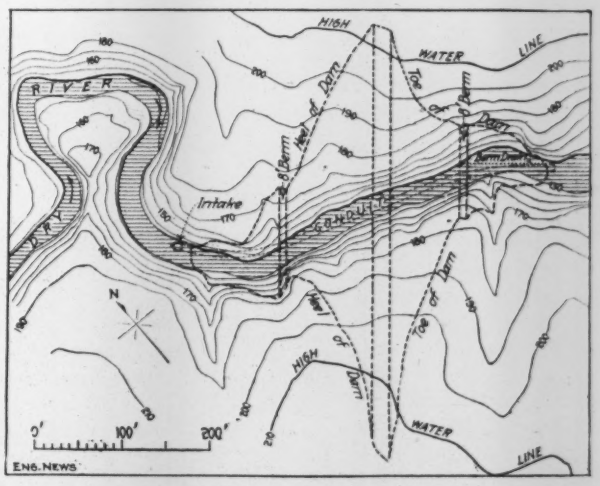


Fig. 2. Plan of Earth Dam

comprehensive plan for the permanent improvement of the stream. The Storm Sewer Commission was authorized by the state legislature, and it engaged as consulting engineers, Solomon, Norcross & Keis, of Atlanta, Ga., to make the necessary investigations.

Improvement Plans—Detail surveys and study of local conditions showed that it was most feasible to create two reservoirs in the narrows of Dry River, west of the city, to release the impounded waters at moderate rates of flow and to carry the stream in a reinforced-concrete conduit built in part along the old stream bed. The two reservoirs would remain empty except in times of freshet, and their principal function would be to retard the ex-

*Solomon, Norcross & Keis, Engineers, Watervliet, N. Y., and Atlanta, Ga.

cess flood discharge. Gas-House Creek was to be turned into a storm sewer, without control works, and led to the main conduit. A short cut was planned to the Hudson, the old bed of the stream below the point of departure being used for a large concrete sewer to carry off the storm water formerly entering the stream from adjoining streets.

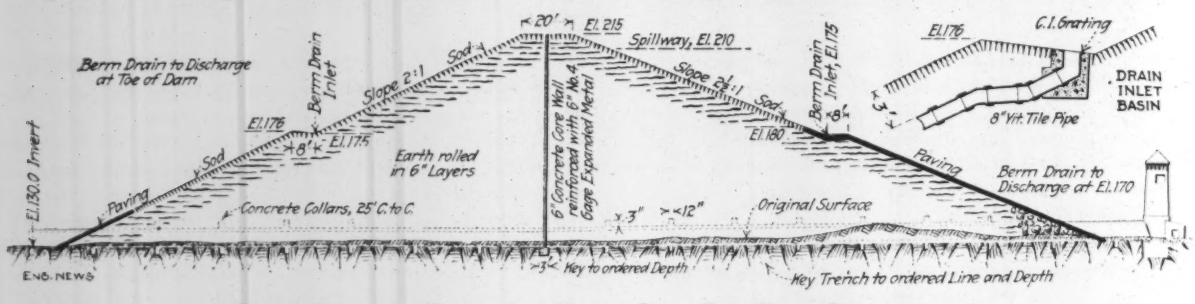


FIG. 3. MAXIMUM SECTION OF EARTH DAM, DRY RIVER IMPROVEMENT

Upper Dam—The upper dam is an earth fill across the upper end of the rock gorge, where the valley broadens out into an open basin. At the damsite the gorge is about 40 ft. wide at its bottom, but increases to a width of 500 ft. at crest elevation, 85 ft. above the bottom. The crest of the dam is 20 ft. wide. The upstream slope is 1 on 2½ and the downstream 1 on 2. Berms, 8 ft. wide, are built at mid-height on both slopes and tile drains carry the drainage to the toes of the slopes. The upstream slope is paved with old street blocks to a point 10 ft. above the berm, while the downstream slope pavement is stopped at 15 ft. above the foot of the slope.

A reinforced-concrete core wall 6 in. thick, on a base 3 ft. wide, was carried down into the rock. The core-wall was built in lifts of from 6 to 8 ft., the concrete work being carried on simultaneously with the fill. Special effort was made to obtain a tight bond between old and new concrete work by means of steel plates.

The earth for the first 50 ft. of embankment was hauled in 4-yd. side-dump cars over a track laid along the edge of the gorge and dumped so that it would slide down upon the fill. Slip scrapers distributed the material in 6-in. layers, after which it was thoroughly rolled by a steam roller and sprinkled. After the 50-ft. elevation was reached, all material was hauled from the bor-

FIG. 4. EARTH DAM AND INLET TOWER; DRY RIVER IMPROVEMENT, WATERVLIET, N. Y.

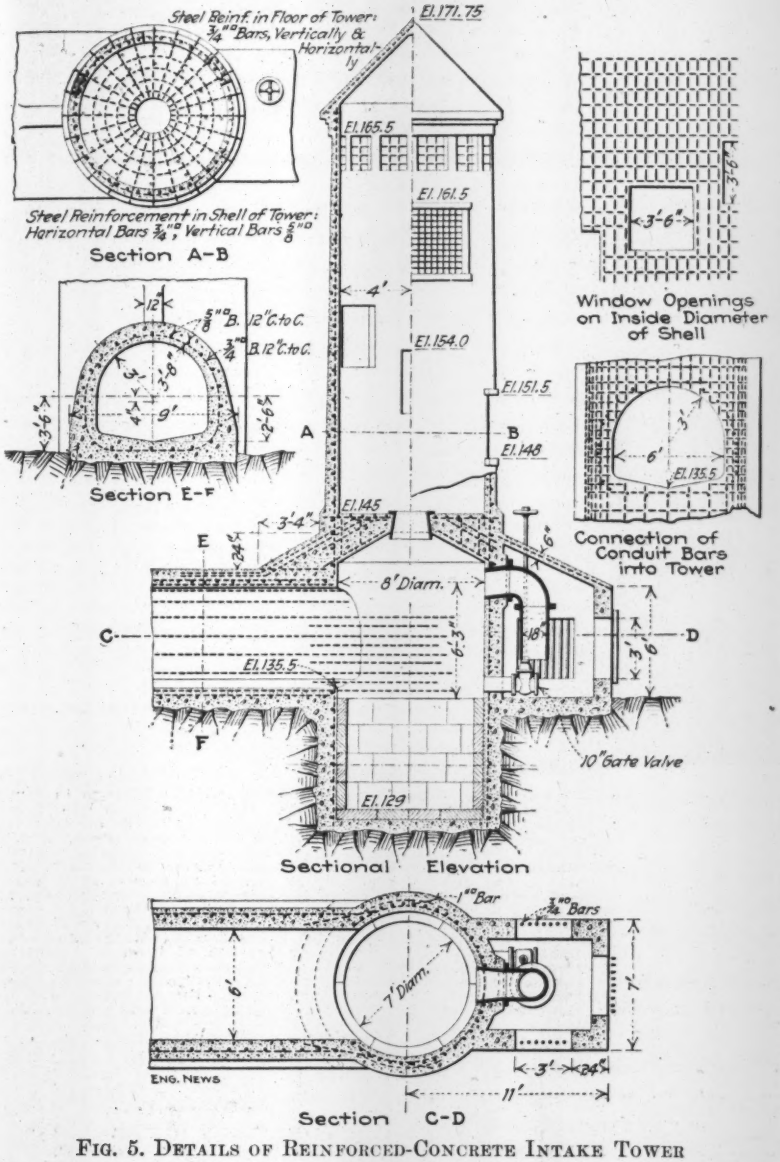


FIG. 5. DETAILS OF REINFORCED-CONCRETE INTAKE TOWER

row pit in wagons and wheel scrapers and compacted as before, the greatest care being taken in the placing of the material for the dam.

INLET TOWER—In order to avoid any moving mechanism and make the operation of the discharge works independent of manual control, a system of orifices, adequately protected against freezing and clogging, was deemed to be best suited for the purpose of regulating the flow of impounded waters. Accordingly, these orifices were placed in a circular tower of reinforced concrete in the positions shown in Figs. 4 and 5. The smallest opening, 12 in. in diameter, is intended for the minimum summer flow. In winter, the 12-in. valve will be closed and the normal flow will be discharged through the 18-in. opening in the side of the tower. When the flow increases above the capacity of this opening, the water will rise to the grated openings set spirally into the tower walls. These openings are five in number and extend to near the top of the tower, where they are capped by a ring of smaller openings extending around the entire circumference. The orifice in the tower floor, which is located above the 18-in. side opening, discharges the water into a granite-lined well, whence it passes downstream to the second reservoir. As the stream flow increases, the reservoir fills until such time as the discharge through the tower equals the rate of runoff. Then the water level remains constant and later it begins to recede. A study of past storms indicates that the reservoir will seldom be filled to any of the higher levels longer than 48 hours.

SPILLWAY FOR UPPER DAM—A depression in the crest of the hills inclosing the upper reservoir forms a natural spillway. This depression opens out into a small tributary of Dry River which enters the stream about a quarter mile below the dam. In the event that the reservoir should ever become filled, the water would flow through this channel to a concrete spillway wall extending across it, discharge upon a paved semicircular apron and be carried thence through an open concrete-lined canal section to the small tributary mentioned.

LOWER DAM—About a half-mile below the earth dam, near the end of the rocky gorge through which Dry River runs and not far from the upper edge of the city plane, is a reinforced-concrete deck dam, 25 ft. high and 105 ft. along its crest. A spillway 72 ft. long is centered on the crest length.

The small reservoir created here receives the discharge from the tower and conduit at the upper dam. The flow from here is regulated in the same manner as at the upper works, though the orifice structures do not much resemble each other. There is, first, a low-lying concrete box with three large orifices for ordinary flows; instead of a tower rising above this outlet chamber, a second orifice box is placed on the deck at a higher level, greater and greater outflow area becoming available as the reservoir level rises.

The ordinary flows are led from the smaller outlet chamber through a 22-in. steel pipe running downstream 1200 ft. to the concrete conduit and entering it about 200 ft. beyond the entrance. The pipe is protected from freezing by lagging on exposed sections. The idea in using the pipe was to prevent the shoals between dam and conduit from freezing over and furnishing ice to choke the sewer intake at spring breakup. The freshet water passes through the deck orifices and is discharged into the stream bed, along which it passes until received by the main conduit.

SEWERS—The concrete conduits are mostly of standard horse-shoe type and range from 6½ to 8½ ft. in diameter. The thickness of walls, arch and floors and amount of reinforcement vary according to the depth and nature of the trench, but for any one diameter of sewer the same steel forms (Blaw) were used and all changes were

made in the outside dimensions. In that part of the work built through solid rock, a special section permitting a narrower trench was used. This section had 8-in. vertical concrete walls, an 8-in. floor sloping toward the middle, and a brick arch for the top.

Some 1200 ft. below the smaller dam, the sewer conduit was started. The entrance was formed by the usual

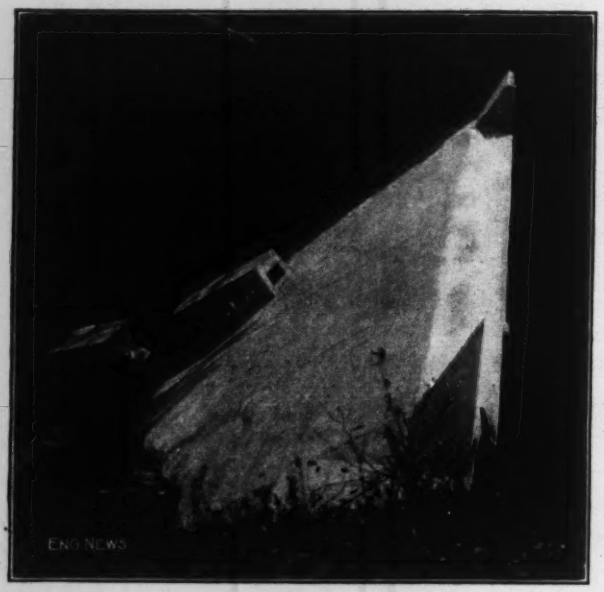


FIG. 6. VIEW ACROSS LOWER DAM, SHOWING OUTLET STRUCTURES

short wing walls converging on the conduit. The stream bed was stopped here by a small earth fill.

The diversion from the old stream bed is at the head of 21st St., some three-quarters of a mile above the old mouth. By carrying the line eastward through this street, the Hudson is reached in 1850 ft. At the sewer inlet the conduit is 6½ ft. in diameter, but this gradually is increased to 8½ ft. to accommodate the city storm water and the flow of Gas-House Creek, which is received at the head of 21st St. This little stream is intercepted by a simple open canal section, on flats west of the central section of the city, going into conduit.

On the cutoff to the Hudson, the conduit has to go under the Erie Canal. The canal bed is lower than the arch line of an 8½-ft. sewer, so that a special section had to be substituted to avoid a siphon. The invert grade was maintained and a twin-box culvert 4 ft. in height was substituted and built so that from the dock lines both ends sloped and converged into the standard section. This is shown in Fig. 7.

The channels of both streams being located for the most part through city streets and developed property in the most desirable sections of the city, are now being rapidly filled in and reclaimed, and all vestige of the streams is rapidly disappearing. During the heavy spring freshet of 1914 the system, although still incomplete, worked very satisfactorily and not a street was inundated.

COST, ETC.—The total cost of the work above described was $165,000. It was planned and executed under the direction of Solomon, Norcross & Keis, the writer being in active charge. Leary & Morrison, of Fairport, N. Y., were the contractors.

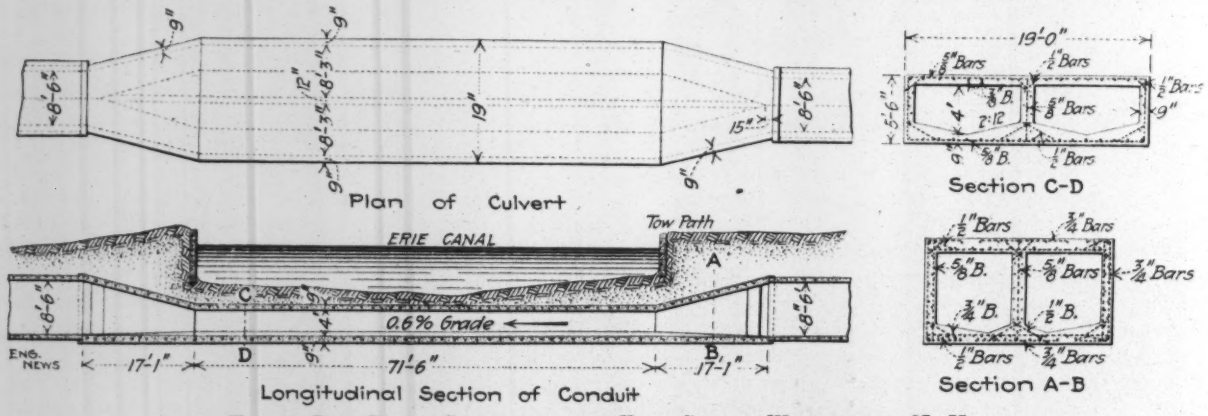


FIG. 7. DRY RIVER CONDUIT UNDER ERIE CANAL, WATERVLIET, N. Y.

✻

The Effect of Hydrated Lime on volume change and strength of portland-cement mortars and concrete has been reported upon by Henry S. Spackman, of Philadelphia, to the National Lime Manufacturers' Association, and the results published in Bulletin No. 10, Oct., 1914, of that Association. Tests were made on mortars and concretes (small gravel aggregate) with varying percentages of both calcium and magnesium hydrate and with the pieces exposed under water and in air, outdoors, and alternately in water and in air. Examination was made to note the expansion of the pieces with variation in moisture content. Curves for all the tests are shown in the bulletin. We give below only the conclusions drawn by the author.

Variation in moisture content as well as change in temperature affect the volume of the test piece; when kept from contact with water other than the moisture in the air there is a marked tendency to shrinkage of the test piece, which tendency continues up to and beyond the six months' period; where the test piece is in constant or frequent contact with water, the general tendency is toward expansion in volume, and with the draining off of the excess gaging water there is a marked shrinkage in the 24-hr. period followed in some cases by expansion at 48 hr. and further shrinkage if test-piece specimens are out of contact with water and expansion if in contact with water; the addition of hydrated lime, while increasing somewhat the maximum expansion and contraction when the test pieces are constantly exposed either to water or dry air, markedly reduces the shrinkage due to the draining off of the surplus gaging water and also the extent of the movement when the test piece is alternately wet and dry.

Further tests for strength indicate that portland-cement mortars and concrete, either with or without the addition of hydrated lime, develop the greatest strength when in continuous contact with water, and where allowed to harden in air out of contact with water there is a marked diminution of strength. Considered broadly, the addition or substitution of 10% of hydrated lime has no marked effect on the strength of the mortars, the test showing increase in strength at some periods and decreases in strength at others.

Editorials

A Novel Dry Reservoir Project

The control of the flashy little Dry River, whose sudden floods have done so much mischief at Watervliet, N. Y., in past years, is one of the interesting pieces of unusual design in hydraulic engineering which have been carried to completion this year. To many the idea will seem natural, as shown in the description of the Dry River improvement elsewhere in this issue, to build works which will store up the freshet waters and automatically let them out at moderate rates of flow within the capacity of the channels. Yet the only other actual employment of the basic scheme that we have seen in municipal improvement work in this country is at the Paxton Creek flood-control works in Harrisburg, Penn. (described in ENGINEERING NEWS, Feb. 17, 1910). The structures and general layout in the two cases are, of course, quite unlike. The same idea, also, is involved in the flood-prevention schemes which involve dry reservoirs scattered in the headwaters of a large stream such as are proposed for the control of the Miami River above Dayton, Ohio.

The Use of Timber on the New York Subway Work

The average New Yorker hardly appreciates the magnitude of the work on the new subway system, because that work is carried on almost wholly beneath the street surface. When the first subway lines were built, the work was carried on in open cut and the streets on which the lines were located were closed to all traffic for long distances, in some cases for many months. The interference with business caused a public demand that in future subway work the street surface must be maintained and kept open to traffic during construction. The original line on lower Broadway was thus built and the same method is being pursued on the new subway lines now under construction in Manhattan and on many of the streets in Brooklyn.

Elsewhere in this issue, F. Lavis describes the methods of timbering used in supporting the street surface on the New York work. The magnitude of this task is one little appreciated save by those directly connected with the work. At Canal St. and Broadway, for example, the subway excavation is a pit some 60 ft. deep and 100 ft. in width. Mr. Lavis estimates that the temporary timbering of the subway cut for 13 miles on Broadway, Lexington Ave. and 138th St. alone has required some 70 million ft. b.m. of timber; and at least twice as much more will be required on the work still to be done.

A rough computation of the cost of this timber and the cost of cutting and placing it shows that this feature alone of the new subways involves an expenditure of many millions of dollars.

It is to be further noted that this method of construction, filling the excavation with a forest of timbers, has made it necessary to do practically all the excavation by hand, and has prevented the use of modern excavating machinery, thereby adding largely to the cost of the work.

No doubt, the construction methods adopted are the best possible under the circumstances; but it is at least an interesting speculation whether a large saving in the cost of construction might not have been made had it been possible to let the work in stretches of five or six miles to a single contractor on broad specifications which would have allowed the development of a type of structure and also of an economical method for its construction.

In Buenos Aires, the capital of the Argentine Republic, a subway line has recently been completed by the open-cut method. The contractors were allowed to work on only three blocks at a time; on one, excavation was being done by a steam shovel, loading the excavated material into standard-gage cars, which were hauled back through the completed structure and up an incline to the surface-car tracks. On the next block the structure, of steel frame and concrete, was being erected and on the third the street surface was being restored. No one block was kept open more than three months, though it should be noted that there was no rock to be excavated. In this case, the contractor (the operating company) both designed the structure and executed the work.

Of course, there are many sections of New York where no interruption at all of vehicle traffic is permissible; but there are many sections where half of the street could be given up, and many others where, were it not for previous experience, the open-cut method might be used.

It is also worth noting that while most contractors on the New York work complete excavation over a large part of their section before beginning to build, others push the construction work simultaneously with the excavation. On Section 1 of Route 5, for example, there are at all times men putting down the street decking, taking out the first lift, underpinning buildings, etc., followed by others completing the excavation, laying the waterproof floor, erecting the steel, concreting, waterproofing and backfilling.

Had this latter plan been more generally followed, it would probably mean a shorter time between breaking ground and completion of any street block, and apparently some economy in the use of timbering, since more of it could be used over and over.

The Fascination of Railway Location

Was there ever any sort of engineering work which gained such a grip upon its devotees (we use the word advisedly) as the work of railway location?

One of the oldtimers, who played an active part in engineering back in the '80's when more civil engineers were engaged on railway location and construction than in any other field of engineering work—a man who served under Wellington in Mexico—confessed the other day: "I know that some engineers when they have a holiday

or vacation like to go off in the country somewhere and play golf; but when I want to take a vacation and get the most enjoyment out of it, I go off in the wilderness somewhere and locate a railroad line."

We doubt not there are thousands among our older readers who look back with fond recollection on their years of work in the field ahead of a construction gang. With all the hardships and not seldom the dangers that accompanied the life of the locating engineer, there was a keen fascination about the work. The study of the topography so as to work out the best line through a difficult country to satisfy required conditions was as much a challenge of the engineer's skill as a game of chess or billiards. This was, without doubt, one of the reasons why, notwithstanding the long working hours, the rough life and the small rate of pay compared with the responsibility carried, there were plenty of railway-locating engineers who stuck to that work in preference to any other field of engineering as long as there was location work to be done.

Hydrated Lime in Concrete

Hydrated lime an as adjunct to portland-cement mortar and concrete is no new thing. Various well known experimenters have shown that small additions of hydrated lime to cement mortars render them practically impermeable to moisture, have no material effect upon their tensile strength and much increase the plasticity of the mortar over that composed of cement and sand alone.

These very commendable characteristics of hydrated lime have been accepted by engineers without much thought as to their application to concrete work. In fact, the makers of hydrated lime, themselves, have not considered it worth while to emphasize any of them except the waterproofing feature, which is unfortunate from a selling viewpoint, because there are many other equally effective ways of making concrete water-tight. It is doubtful, however, if one engineer in twenty, accustomed to the use of concrete, has an adequate appreciation of the possibilities of hydrated lime, due to those very qualities upon which many experimenters have reported.

In two fields of concrete work there would appear to be a distinct advantage in using hydrated lime; in concrete which is to be delivered by the spouting method and in concrete roads. In the former case it has long been a cause for complaint that while spouting is a most, perhaps the most, economical and effective way of getting concrete to the forms, unless the tower is exceedingly high or the radius of deposition quite short, the low angle of slope requires an excessively wet concrete to insure easy spouting, and every experimenter knows that an excessively wet concrete is a dangerous concrete. It seems probable, however, that a proper admixture of hydrated lime would give the concrete a "buttery" consistency, a certain slipperiness, as it were, which would lower its friction coefficient and permit of its spouting on a lower slope. Some few attempts to utilize this advantage have been successful, but they have only been extensive enough to show that there is here a field for investigation and practice well worth pursuing.

The desirability of hydrated lime for concrete road work was well advocated at the recent Road Congress at Atlanta by a lime manufacturer, Charles Warner, of Wilmington, Del. A point in favor of the use of lime in road concrete emphasized by Mr. Warner is the effect of the hydrated lime in reducing the tendency to segregation in the wet mixture of cement, sand and stone, and also in reducing the ratio of the expansion when the concrete changes from a dry condition to a wet condition. This reduced expansion is doubtless a result of the decreased permeability of the concrete made with hydrated lime. The importance of reducing segregation ought to be apparent without argument. One of the difficult things in laying a concrete road is to get the mix wet enough so that it can be readily tamped into place and will form a solid mass without voids and yet not so wet that the cement and sand will separate as the water flushes to the surface. The addition of hydrated lime materially decreases the tendency of the cement and sand to separate under the screed or the trowel, and at the same time makes the mass flow more readily into place.

It might be thought that mixing the lime with the cement, even though it did not affect the tensile strength of the mortar, might alter its hardness, since lime-mortar is much softer than cement. Since there is no place in which concrete is used where extreme hardness and resistance to concussion and abrasion are of greater importance than in a concrete road surface, the probable wear of a hydrated-lime concrete road is equally important as the ease of construction. Here is something that experience alone can answer. We are informed that at least one concrete road is this year under construction with the admixture of hydrated lime; its behavior will be watched with interest by engineers.

Of course, it can readily be understood that the hydrated lime must be very thoroughly mixed with the cement to secure the results above set forth. If such thorough mixing is not secured and some parts of the mortar as laid in the completed work have a deficiency of hydrated lime while others have an excess, the strength of the work might be considerably affected. The question how practically to effect such thorough mixing is not as easy as might at first sight appear. Where mortar alone is to be used and the machine mixer is available, it will be comparatively easy to put the right proportion of hydrated lime in each batch and keep the mixer turning until the lime, cement and sand are thoroughly incorporated. The great bulk of portland cement, however, is used in making concrete and the accurate proportioning of the lime to the cement and its thorough mixing with the stone and sand are matters deserving some study.

Indeed, practically all of the advantageous features set forth above have had only limited practical proof. Theoretically and experimentally there is much to be said in favor of the proper use of hydrated lime in concrete. Therefore engineers will look for reports on the actual construction which the next year will undoubtedly bring forth.

Salt Water in Miraflores Lake on the Panama Canal: A Curious Hydraulic Paradox

A very remarkable hydraulic problem has recently been solved on the Isthmus of Panama, and solved by the records of experience in a manner quite contrary to the anticipations of the engineers. The facts concerning it are briefly recited in the Annual Report of Colonel Goethals, Governor of the Canal Zone, just issued. In what

follows, we have added enough to the statements given in the report to make the matter clear to those unfamiliar with the topography of the Canal Zone.

A year and a half ago, the question of a source of water-supply for the towns at the western end of the Canal, including the terminal at Balboa, was under consideration. Various projects were considered, but the one involving the least cost of any was to take the supply from Miraflores Lake. A further advantage of this supply was that its quantity was sufficient for every requirement of the future.

Miraflores Lake is a basin a little less than a mile long, extending from the foot of the Pedro Miguel Locks (which are at the Pacific end of the Culebra Cut) westward to the head of the Miraflores Locks. The water-supply of Miraflores Lake comes from Gatun Lake through the Culebra Cut. Every time a vessel passes through the Pedro Miguel Locks, a lockful of water is discharged into Miraflores Lake, and a similar amount is, of course, abstracted from Miraflores Lake as the vessel passes through the Miraflores Locks. There are also some small streams draining into the lake, and it is, of course, possible at any time to supplement the flow into the lake from Culebra Cut through the Pedro Miguel Locks.

The surface of Miraflores Lake is 55 ft. higher than mean tide in the Pacific. There are two locks in the flight at Miraflores, dividing the lift into approximately two equal parts, although the lift in the lower lock is variable, according to the state of the tide. The extreme range of the tides at Panama is about 20 ft. Besides this, it is to be noted that the Miraflores Locks are about 8 miles inland from the Bay of Panama, on which the terminal at Balboa is located. There is practically no current in the excavated channel at sea level from Miraflores to the Bay, except such as is created by the discharge of water from the locks. The tide merely flows and ebbs; but every time a vessel passes Miraflores a lockful of water is discharged into the channel, and this amount of water, unless the vessel is locked through with the basin part full, means a prism 1000 ft. in length, 110 ft. in width, and some 28 ft. in depth at mean tide. Further than this, the bottom of Miraflores Lake is at a higher elevation than high tide in the sea-level section below the locks.

We have recited all these conditions to show that the engineers who made the decision to use Miraflores Lake for a water-supply had no reason to suspect, so far as we can see, that salt water from the Pacific would invade Miraflores Lake in any material quantity. The possibility that this might occur was, indeed, canvassed. The official report says:

> **At the time that the use of Miraflores Lake water was considered, the possible objection was advanced that the chlorine content, by reason of the operation of Miraflores Locks, might increase beyond 75 to 100 parts per million. At the time, it did not seem possible that this would occur, at least for a period of years, on the assumption that intimate diffusion between the salt water admitted by the locks and the fresh water of the lake would not be rapid, especially in view of the fact that water could be pumped from one of the fresh arms of the lake. At any rate, the enormous saving that would result seemed to warrant adopting the Miraflores Lake project.**

To the great surprise of all parties concerned, however, salt water from the ocean invaded Miraflores Lake at an astonishing rate, even though there was very little operation of the locks at that time. Chlorine-sampling stations were established at the lake in January, and by February it became apparent that a constant diffusion of salt water was taking place throughout the whole area of the lake, extending back into its arms. The proportion went as high as 15% of salt water. To meet the emergency, about 4000 gal. per min. was drawn from Culebra Cut above the Pedro Miguel Locks, and discharged into Miraflores Lake in front of the water-supply pumping station. This gave a supply of fresh water, but also a muddy water, due to the conditions in the cut.

It was obvious, of course, that with this large amount of salt water already invading the lake it would be impossible to use it as a source of water-supply. It was therefore abandoned, and a pumping station was set up on the Chagres River at Gamboa, requiring laying a line of 30- to 36-in. cast-iron pipe a distance of some ten miles. Before this radical action was taken, an attempt was made to clear Miraflores Lake of salt water by drawing out the water in it through the Miraflores Locks and admitting fresh water in its place through the Pedro Miguel Locks, but the results were not satisfactory.

Now comes the interesting engineering problem: How does this salt water from the Pacific gain access in such quantities and with such rapidity to this lake situated eight miles distant from the ocean, and with its surface 55 ft. above mean sea level, and its bottom also higher than the level of the ocean? The only explanation possible, of course, is the diffusion of the water in the process of lockage.

Suppose a vessel is to ascend the Miraflores Locks. She enters the lower lock and the gates are closed behind her. She is then floating in a mixture containing perhaps 75 per cent. of salt water from the ocean. The valves are then opened, and fresh water from the lake above is admitted into the lock through the openings in the bottom until the level of the water in the lock is raised to the level of the water in the upper lock. In this process of filling the lock, the salt water and fresh water are thoroughly mixed. The water in the upper and lower locks being now on the same level, the gates between the two are opened and the vessel is moved into the upper lock. While this movement is taking place, however, the heavier salt or brackish water in the lower lock flows into the lower part of the upper lock by reason of its greater specific gravity, while the lighter fresh water in the upper lock flows rapidly over the heavier water in the lower lock. In this manner, while the vessel is being transferred from one lock to the other, a considerable quantity of brackish water enters the upper lock.

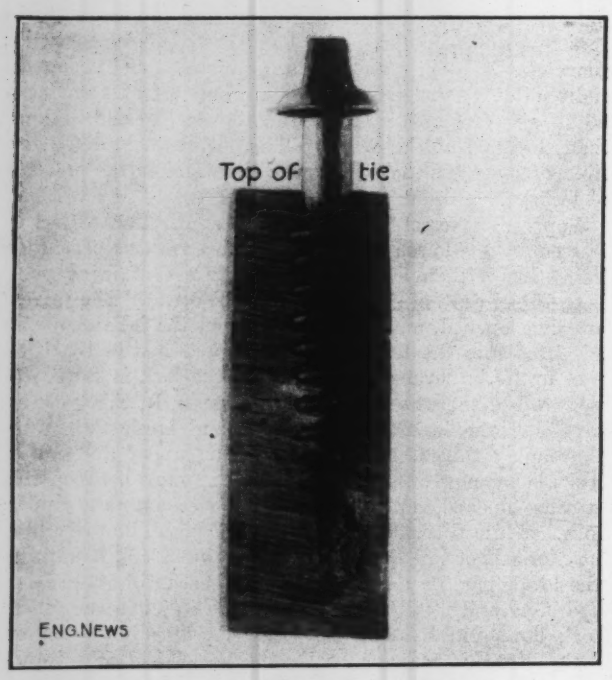
The gates are then closed behind the vessel and fresh water from the lake is admitted through the bottom of the upper lock, until it is filled to the same level as the lake. By this time, certainly, the percentage of salt water in the upper lock should be reduced to a very small amount. Yet there is evidently enough there so that when the gates are opened and the vessel passes out into the lake, a considerable amount of the water in the upper lock chamber flows out with it, and is replaced by entirely fresh water from the lake.

A question of some interest is whether increased traffic will cause an increased amount of salt water in the lake. While it will cause frequent repetitions of the process above described, it will, on the other hand, bring a larger quantity of fresh water down from Gatun Lake to Miraflores, and will also discharge a larger quantity from Miraflores Lake into the channel below.

Letters to the Editor

Screw Spikes and Auxiliary Rail Fastenings

Sir—An article in your issue of July 16 describes some experiments with screw spikes and auxiliary rail fastenings on the Pennsylvania Lines, being in part compiled and quoted from Bulletin No. 165 of the American Railway Engineering Association (March, 1914). This article omits mention, as does Bulletin No. 165, of common defects in the insertion of screw spikes with Thiollier helical linings, which defects are illustrated by the full-page engraving on p. 300 of the Bulletin and of the 1914



Screw Spike with Helical Lining, Showing Faulty Position of the Lining in Relation to the Threads of the Spike

Proceedings of the Association. [This is shown herewith.—Editor.] These defects are the following:

1. Threading of the tie to receive the helical lining so that the helical linings are not to the same pitch as that of the screw-spike threads. It is very difficult to accomplish this perfectly, and probably a slightly different threading tool is needed for kinds of wood differing in hardness and general character.

2. The helical lining engages the lower part of the screw spike. For good results, the lining should be longer, to reach from bottom of screw spike to close to the top of the tie. If the lining is of present length, the tie should be threaded only deep enough to cause the helical lining to have its top very near the top of tie, the screw spike then being driven through and projecting below the kink in the bottom of the helical lining, and the spike threads below that point thoroughly engaging the uninjured wood of the tie. By this method, there is less tendency to lateral movement of the spike and very greatly reduced tendency for water to enter the tie and rust the spike and the lining.

3. The cut shows the screw-spike threads on the upper side of the threads of the helical lining and pulling against very much injured wood, instead of the spike threads being where they ought to be, close underneath the threads of the helical lining and pulling against them, as they are designed to do.

Apparently a screw spike inserted as illustrated has much less effective permanence than if placed in the tie without any helical lining and without the defacement of the tie by the threading for the lining. It is worth while to call particular attention to this matter, for the reason that if the screw spike is inserted without special care to start the spike threads under the helical lining, the chances are several times to one that the screw-spike thread will run on top of the helical lining, as shown.

It seems likely that a very large percentage of all screw spikes with helical linings now existing in railway tracks experimentally or otherwise have these defects.

William Hood.
Chief Engineer, Southern Pacific Ry.
San Francisco, Calif., Oct. 30, 1914.

Comparison of Results of Paying for Public Improvements with Bond Proceeds and from the Tax Levy

Sir—In catching up with my professional reading, I note you mention the latest financial policy of the City of New York.

For some time I have been strongly advocating a method almost exactly similar to that now adopted. In this connection a memorandum was prepared analyzing some of the arithmetical differences in the old and the new methods. A copy of such memorandum is attached. Some of your readers may find arguments which can be modified to meet special conditions in municipalities where the old methods may perhaps still be in force.

E. P. Goodrich,
Consulting Engineer to the Borough of Manhattan.
New York City, Oct. 17, 1914.

[Presumably Mr. Goodrich's notes will interest and they may at some juncture assist many of our readers. Case I shows how a modest "pay-as-you-go" plan will give a considerably larger number of "improvement years" than a heavy bond issue and also shows the relative effects of the two plans upon the taxpayer. Case II shows how 50-year bonds may adversely affect the finances of a city having both a bond and a tax limit. The memorandum follows:

CASE I

Assume 5% for interest and sinking-fund requirements on 50-yr. bonds.
Assume $1,000,000 available annually to finance public improvements.

The $20,000,000 issue of 50-year bonds would provide public improvements immediately (assumed as one year after the bonds were issued). The annual interest on this $20,000,000 at 5% amounts to the same as the $1,000,000 assumed to be annually available for improvements to be paid in cash. Thus, 20 improvements at $1,000,000 each can be enjoyed for 49 years. But if the initiation of improvements is delayed so that only one is initiated each year to the extent of $1,000,000 and cash paid therefor, improvements would be secured and enjoyed as follows:

One improvement would be secured the first year and enjoyed for 49 years.
One improvement would be secured the second year and enjoyed for 48 years, etc., etc., etc.

The 20th improvement would not be secured until the 20th year, and it would be enjoyed for only 30 years; but at the expiration of the 50-year period, 49 improvements would have been secured and enjoyed on an average of 24½ years each; while one additional improvement would have been secured during the 50th year. This makes a total of 1200 "improvement years." In the case of the bond issue, there would have been only $20 \times 49 = 980$ "improvement years." At the end of the 50 years, the cash method would have secured 225 additional "improvement years." This additional enjoyment would obviously have been delayed until toward the end of the 50-year period; and the question to be answered is whether the additional delayed improvements are worth more than the immediate enjoyment of a less number of improvements.

Total expenditure for improvements by bond method	$20,000,000
Total expenditure for improvements by cash method	50,000,000
Total payment by taxpayers, each method	50,000,000
Interest paid to bankers, bond method	30,000,000

COMPARATIVE EFFECT ON TAXPAYER—It is often stated, as against the cash method, that a taxpayer has the use of his money in his own business during the 50-year period; and that when he pays cash for an improvement, he is thereby deprived of certain profits which he might have made.

Assume 5% for interest and sinking fund on 50-year city bonds.
Assume that an individual taxpayer's share of an improvement was $100.

Obviously, his share of the interest and sinking-fund requirements would be $5 per annum over a period of 50 years. In other words, he would have the use of $95 of his income compounded over that period. On the other hand, if he paid $100 cash the first year, he would have the use of $100 of his income compounded annually for 49 years. The accumulations at the end of the 50-year period are easily computed mathematically by well known actuarial methods, the formula being annual sum involved × ([(one plus the interest rate) raised to a power equal to the bond period] minus one) divided by the interest rate measured in hundredths. Making these computations on the assumption that the taxpayer can compound his money at the 10% rate per annum in each case, the problem resolves itself into $95 compounded for 50 years against $100 compounded for 49 years. The totals in the two cases are, respectively, $110,572 and $105,720. There is a difference, therefore, in favor of the bond method as far as the taxpayer is concerned, of $4852 accumulation at the end of 50 years. This is a matter of only four and a fraction per cent., which is to be offset against the enjoyment of a greater number of improvements where cash is paid for them.

In this connection, it is to be recalled that the majority of taxpayers are wage earners or salaried employees, and that few of them have opportunities or the ability to compound their savings at any such rate of interest. To the large majority of taxpayers, therefore, the problem is not one of a 10% compounding, but only the interest which saving banks will pay on deposits, or which may be realized from absolutely safe investments, such as are always recommended for the man with small earning power. If the above computations are made on the basis of a 5% compounding, the results are, respectively: $19,888 accumulations if bond method is used, against $19,843 if cash is paid for the improvement. This is an insignificant item. Even in the case of a 20% compounding, the taxpayer would have accumulated at the end of 50 years only about 12% more than had cash been paid.

CASE II

Assume a 10% limit of bond indebtedness for the municipality.
Assume a 2% limit in the tax rate for such municipality.
Assume 5% as the interest and sinking-fund requirements on 50-year municipal bonds.

With a valuation of $1000, the bonds outstanding would amount to $100. The eventual interest and sinking-fund requirement would be $5, while the total taxes would be $20. In other words, 25% of the total tax rate would go for interest on indebtedness. The other 75% would normally be expended for general municipal administration. If it be assumed that the full limit of indebtedness is maintained at all times, so long as the municipality is growing at a constant rate per annum (geometrical ratio), improvements can be made through and paid for by bond issues to the extent of 10% of the yearly increase in valuation. Should this increase stop for any cause, public improvements must similarly cease. Under such circumstances, no increase can be made in the tax rate for improvements or for increase in cost of administration. In other words, the municipality is absolutely estopped from developing itself so as to bring about an increase in valuation through the effect produced by public improvements. It has throttled its own growth. Had the "pay-as-you-go" method been adopted, more improvements could have been secured in any period longer than that measured by the bond interest rate divided into 100, and, any time, the community would have been in a position to expand or contract its tax rate (within the assumed 2% limit above mentioned) as community demands required. An elastic method would have been secured with the corresponding beneficial result.

A point which Mr. Goodrich might have made is that although under his "pay-as-you-go" plan a considerable number of the "improvement years" are deferred, a considerable part of the improvements would still have many years of usefulness at the end of the period.—EDITOR.

Questions in Building the Embankment of the Yale Bowl

Sir—The statements contained in your editorial, "The Yale Bowl," in the last issue of ENGINEERING NEWS, taken in connection with the description of the work by Mr. Atwood, in the same issue, conveying an entirely erroneous impression as to the proposed method of building the Bowl as planned by the writer. The editorial says:

The original project contemplated making the embankment a single dump, without much or any attempt at consolidation; the material was to be deposited in 8-in. layers and not rolled. The contract and specifications were actually drawn on this basis. Of some interest in the same connection is the fact that it was originally planned to build the concrete seating structure all the way up the slope, on the fill as well as on the part in cut, and this with unconsolidated fill as originally contracted for.

The following is an extract from the original specifications as drawn by the writer:

The embankment shall be built to the exact shape shown on the plans, the earth being deposited in horizontal layers not more than 6 in. in thickness, and be solidly compacted by sprinkling with water and rolling with rollers weighing at least 400 lb. per foot of length. Two or more rollers shall be used as may be necessary; one a grooved roller and the other a smooth one. They shall be used together, one following in the track of the other. The embankment on the side toward the field shall be kept "full" of the theoretical lines at least 1 ft. to permit of the earth being rolled out to the edge of the embankment, and to afford a solid foundation for the concrete slabs. The tunnels shall be constructed as soon as the embankment has been built up to the level of the top of them.

With an embankment constructed in the manner specified, the writer believed that it would be perfectly safe to lay the concrete facing directly on the sand, on the filled portion of the slope as well as on that in cut. The Consulting Engineer, however, thought otherwise and maintained that it was impossible to consolidate sand by rolling; that such an embankment could only be made solid by the action of the elements acting through several years. To meet this view, he devised a plan of concrete facing using reinforced slabs supported by reinforced-concrete girders resting on piers built into the face of the embankment; any irregularities in the facing, due to settlement, to be taken care of by, occasionally, shimming

up the girders. As it would be somewhat cheaper to place the material in 18-in. layers without rolling than by the method originally specified, the specifications were therefore changed by him to the form mentioned in your editorial.

When the Advisory Engineer was appointed, the question as to the best method of placing the concrete facing on the slope was, with several other disputed points, referred to him; whether in the form of a solid slab of concrete deposited directly on the earth, as intended by the writer, or in the form of an elevated structure raised slightly above it, as proposed by the Consulting Engineer. The decision favored the solid slab, and the Consulting Engineer thereupon severed his connection with the work.

The Advisory Engineer then made a supplementary agreement with the Contractor in which the specification for making the embankment was, substantially, the same as originally drawn by the writer except that the weight of the rollers was doubled.

In the meantime, most of the tunnels had been constructed, so that the new specification only applied to that portion of the embankment which was above them.

A comparison of the volumes of the cut and fill shows that the material in the embankment is nearly 7% more dense than that in the natural bank.

After the embankment was finished, last June, thirty benches were established around the top. Recent levels show that the greatest settlement at any of these points has been about $\frac{5}{16}$ in., the average of the whole being about $\frac{3}{16}$ in.

Whether any settlement which is likely to take place will be injurious to the structure is a debatable question, but it is believed, by the writer that, as the blocks are heavily reinforced to prevent cracking, and are "shingled" on to the slope—i.e. each block has a 3-in. overlap on the block below—no serious injury would have ensued if the concrete facing had been completed without waiting for further settlement. He believes that the opening between blocks due to temperature changes is likely to exceed that resulting from any settlement.

CHARLES A. FERRY,
Designer of the Yale Bowl

New Haven, Conn., Nov. 18, 1914.

NOTES AND QUERIES

An 8-Ft. Waterway in Illinois—In the note in "Engineering News," Nov. 12, p. 1000, describing the proposed new 8-ft. waterway from Chicago to the Illinois River, for "sets of levels" read "sets of locks." The number of locks will be eight instead of five, including a lock connecting with the drainage canal at Lockport, and their total lift will be about 140 feet.

In the description of the power saw for cutting timber in our issue of Nov. 19, 1914, p. 1030, the address of the maker was omitted. The machine is made by the Whitman Agricultural Co., St. Louis, Mo.

Statistics of Pipe Production in the United States, compiled by the American Iron & Steel Institute, show that in 1913 about seven times as much steel pipe as iron pipe was made. In 1905 the amount of steel pipe was only a little more than double the amount of iron pipe. The amount of iron pipe produced year by year, however, remains about stationary or slowly declines, while steel pipe production has rapidly increased. The use of steel in making welded pipe was not successful commercially until about the close of the '80's but has had a very rapid growth in the quarter century since it began, the output last year having been 2,189,000 tons.

Unsatisfactory Performance of Brass and Bronze on the Catskill Aqueduct*

Many valves, pipe fittings, bolts and other metal objects large and small on the Catskill Aqueduct for New York City were made of bronze or brass wholly or in large part. These alloys were selected for objects to be exposed to corrosive influences where durability was most important because of the difficulty of replacement or inspection. Nevertheless, by far the greater number of sluice gates, valves, pipe fittings and similar objects were made of iron and steel, in accordance with usual practice. On account of the magnitude of the project, however, unprecedented quantities of bronze or brass were used—a total of nearly 3,000,000 lb. About two-thirds of the total consists of castings, and a large part of the remaining million pounds consists of forgings. The remainder comprises rolled plates, rolled and drawn rods, drawn and extruded shapes, etc. The individual pieces range in weight up to 22,000 lb. of castings, and up to 3200 lb. of forgings.

Unsatisfactory results have developed in the case of the cold-worked material (drawn or cold-rolled), and some of the smaller hot-worked pieces. A. D. Flinn and E. F. Jonson (Inspector) made the matter their special study from the time the first defects were found. The study has not up to the present indicated where the responsibility for the trouble rests, but the inspection has been very thorough and successful, every effort being made to find and eliminate defective pieces.

The word "defective," in this connection, describes pieces which appeared satisfactory when made and furnished and whose material met the tensile tests prescribed in the specifications, but which after a time developed cracks.

The larger part of the total of brasses and bronzes used on the Aqueduct consists of so called "manganese bronze." A considerable amount is Tobin bronze and other brasses or bronzes of related composition. These mixtures are of high strength, so far as shown by the standard tests, and were chosen because of the high requirement on some of the parts in question as to loads and stresses, and the necessity for using a metal which would not corrode appreciably in moist atmosphere or water.

The specifications required that manganese bronze should have:

> A tensile strength of not less than 65,000 lb. per sq.in., an elastic limit of not less than 45% of the ultimate tensile strength, and an elongation of not less than 25%.

The general run of the acceptance tests was about as follows: *Castings*—ultimate strength 66,000 to 73,000 lb. per sq.in.; yield-point 33,000 to 50,000 lb. per sq.in.; elongation 25% to 40%; fracture granular to silky, the breaks frequently being cup-shaped. *Forgings*—ultimate strength 68,000 to 77,000 lb. per sq.in.; yield-point 36,000 to 52,000 lb. per sq.in.; elongation 31% to 41%; reduction of area 43% to 47%; fracture frequently silky.

It must be noted, however, that the yield-point in such material is not sharply defined as it is with structural iron or steel. Moreover, maximum service stresses were allowed by the specifications to go as high as two-thirds of the yield-point in some extreme cases.

Bearing further on the specification requirement, it is

---

*Based on statements by A. D. Flinn, Deputy Chief Engineer of the Board of Water Supply, New York City, in a recent address before the Brooklyn Engineers' Club.

to be noted that pipes were required to stand flattening the end to a separation not greater than twice the thickness of the metal, without cracking, and a piece of the pipe 3 in. long was required to stand splitting and opening out flat without cracking, in both cases after annealing.

Cracking Discovered

Cracking was first found in some bolts and rods installed in a valve chamber, not yet in service, but the examination inspired by this circumstance soon led to the discovery of quantities of cold-worked material such as large plates, bolts, rods, ladder parts, etc., which were in the storehouse awaiting installation, some having been there for months. The pieces showed cracks ranging from superficial to deep, in the cylindrical parts usually circumferential, extending part way or all the way around; a few pieces had actually broken apart of themselves. Some specimens did not show cracking on the first examination, but developed it later.

Extent of the Trouble

The trouble was first noted in cold-worked metal only. Later, however, hot-rolled rods, small forgings, plain extruded (hot) rods, etc., also showed failures "in disturbingly large quantities." The same statement applies to cold-worked brass and bronze rods which had been thoroughly *annealed*. No trouble was experienced with castings and with large forgings, and fortunately these classes cover the largest amount of the brasses and bronzes on the aqueduct.

Summarizing as to the extent of the damage: Large numbers of brass bolts (the term "brass" includes bronze) have been found cracked and broken in their packing cases after storage through a winter without ever having been stressed; others, never exposed to low temperatures and never stressed, have also been found cracked and broken. This statement applies primarily to bolts of ½- to 2-in. size, but it is also true of flat bars, plates and long rods. It applies, moreover, to ¼-in. flanged plates, carefully inspected after manufacture and found in good condition, which some months later showed pronounced cracking and worthless rivets. No brand or make of brass or bronze tested has wholly escaped. In short, the failures were so extensive and numerous (for instance, hundreds of bolts broke at one time or another under tension, heads came off, etc.) as to cause the gravest apprehension and lead to the substitution of steel for brass in a great many cases, in spite of the corrosion trouble with steel.

Attempts to Detect the Cause

Similar trouble has been observed elsewhere in minor degree, and has been studied scientifically. Martens and Heyn, of the German government testing laboratory at Gross-Lichterfelde, concluded that excessive initial stress was usually responsible, and recommended brushing the surface of suspected brass or bronze with a solution of a mercury salt, which in a few minutes would develop cracks if the surface of the brass was under great initial stress. The Aqueduct material also was found to have high initial stress in cold-worked pieces, the surface being usually in tension. But trials of the mercury test were not very satisfactory: many rods which showed no cracking from the mercury solution cracked subsequently, or other rods of the same lot cracked. Further, such a test can be applied to only one rod out of a lot or to only a small part of the surface of a piece, so that even if certain in its results it would not be useful for regular inspection.

Trials of the scleroscope were also made. This well known instrument measures surface hardness. It proved useful, but not sufficient in itself to discover whether a given lot of brass is subject to the deterioration in question. The surface of cold-worked metal being harder than normal metal, the scleroscope indications tend to show whether and how severely a piece has been cold-worked.

General Result of the Studies

A vital question which results from the above recounted experiences is whether brasses of high tensile strength can be produced in such a way as to form a reliable structural material, and what composition or other requirements must be specified in calling for such brass. An equally important question is what tests can be used to detect the defective quality of a piece with regard to the cracking above referred to. A subsidiary question is what safe working stress may be used in designing with high-strength brasses or bronzes. The engineering department of the New York City Board of Water Supply considers it impossible to answer any of these questions at the present time.

The Annual Report of the Panama Canal Operations

The annual reports of the Isthmian Canal Commission, which have appeared regularly since the United States Government assumed control of the work, are now completed with the report for the year ending June 30, 1914, just made public. The present report, in fact, is made by the Governor of the Canal Zone, Col. Geo. W. Goethals, the official existence of the Isthmian Canal Commission having been terminated on Apr. 1 last.

Under the present organization, the Governor of the Zone is in charge of the Department of Operation and Maintenance of the Canal, assisted by an engineer of maintenance and a superintendent of canal transportation, these positions being now held respectively, by Col. H. F. Hodges and Capt. Hugh Rodman, U. S. N. To carry on the remaining construction work as well as work in connection with maintenance and operation, a division of terminal construction was organized in charge of Admiral H. H. Rousseau, U. S. N. The work previously done by the Quartermaster's department and the Subsistence department, is now in control of the Supply department under Capt. R. E. Woods. Other divisions of the organizations are an accounting department and a health department. Work in connection with the civil government of the Zone is under charge of an executive secretary.

The past year has been notable for the practical completion of the construction work. The amount of concrete laid in the Gatun locks aggregates 2,067,731 cu.yd., and the average cost has been $7.21 per cubic yard. Concerning the Gatun dam, whose safety and water-tightness were the theme of so much discussion in engineering circles seven or eight years ago, the report states that seepage from the dam has been negligible. At the close of the rainy season two small streams were found issuing from the north toe of the dam in the west portion, but with the advance of the dry season these ceased entirely.

In the east portion of the dam there was no seepage of any kind apparent.

In the Pedro Miguel locks the total amount of concrete laid has been 928,326 cu.yd., and the average cost was $5.06 per cubic yard. In the Miraflores locks there has been placed 1,507,794 cu.yd., at an average cost of $5.17 per cubic yard. The entire cost of the gates for all the locks has been a little under six and one-half million dollars, of which the contractor for the gates received $5,633,000, and the balance (a little less than $840,000) was paid for inspection, painting, fixed steel, etc. The action of the gates in service has been entirely satisfactory, the leakage at the quoin and miter posts being extremely small.

The report gives information of especial interest relating to the experiences with the paint with which the lock gates are covered. Great pains were taken in the specifications for the painting and in the application of the paint, three and four coats being used, the materials being red lead, graphite, United States Navy anticorrosive, and anti-fouling paint, a damp-proof paint, and a hydro-carbon mixture. An abstract of this part of the report appears in the next column under a separate heading.

The machinery installed for the operation of the locks has cost a very large sum of money. A few of the items are as follows: Gate-operating machines, $822,000; electrical appliances for gate machines, $207,000; rising-stem valves and guard valves and machines, $1,509,000; cylindrical-valve machines, $228,000; chain-fender machines, $831,000; spillway gates, $338,000; towing tracks and material, $1,182,000; towing locomotives, $527,000; hydro-electric plants, $628,000; high-power transmission line, $1,014,000. Very considerable sums must be added to these totals for the substations and other items of machinery and electrical equipment. The emergency dams above the locks at the summit level have been completed and tested. As many of our readers will recall, they are similar in general design to those used in the United States ship canal at Sault Ste Marie. They are intended to provide a means for shutting off the flow of water through the locks in case by some accident the lock gates should be destroyed in the upper lock, allowing a continuous flow of water from Gatun Lake. A test was made of the emergency dam at Gatun in May last. With a full head of 47⅔ ft. of water against the emergency dam, it was found to have a leakage of 950 cu.ft. per second. This leakage did not produce a dangerous current in the locks and it would have been easy to close any of the lower gates in the face of the stream. It was evident, however, that the emergency dam could not be used instead of a caisson for unwatering the locks. Experiments are being made to devise a means of stopping the leakage.

Three central air-compressing stations at Empire, Balboa and Rio Grande were operated during the year, with outputs and cost as follows:

Plant	Output, Cu.Ft.	Cost per M Ft.
Balboa	2,739,000,000	$0.048
Empire	2,908,000,000	0.035
Rio Grande	372,000,000	0.028

Earthquake shocks at Panama have been more numerous than in any year since the American occupation. Eighty-seven distinct shocks were recorded at Ancon. Two shocks during the year were severe enough to throw the recording pens of the seismograph off the record. One of these shocks caused slight damage to the new Administration Building under erection at Balboa. No other damage was done to the canal works.

The steam-shovel excavation in Culebra Cut to June 15, 1914, aggregated 110,262,000 cu.yd., at an average cost of 70.66c. per cu.yd. Of this amount over 25,000,000 cu.yd. were removed because of slides. Of the dredging work done in Culebra Cut since water was admitted, 3,432,000 cu.yd. were removed during the year, of which 919,000 cu.yd. were earth. The average cost was 52c. per cubic yard.

The total net cost of the Panama Canal to the government of the United States, including the amount paid to the French canal company and the Republic of Panama and including the appropriation made Aug. 1, 1914, now stands at $353,559,000.

Experience with Paint on the Panama Canal Lock Gates

The enormous steel gates on the Panama Canal locks have cost the government nearly six and one-half million dollars. The lower portions of these gates are immersed in water. In the lowest locks, the water contains a large percentage of salt from the ocean. The fresh water in Gatun Lake, which flows through the locks, moreover, is heavily charged with vegetable acids from the extensive swamps submerged by the lake.

Under these conditions it was realized in advance that the protection from corrosion of the portions of the gates under water would be a very severe test of the paint applied to the gates. Moreover, those parts of the gates which are above water are exposed to much more severe conditions than exist in the climate of the temperate zone on account of the extreme heat and moisture which prevail at Panama.

Engineers interested in the preservative qualities of paints, therefore, will be much interested in the annual report of Colonel Goethals, Governor of the Canal Zone, just made public, in which the experience thus far obtained with the paints on the lock gates is summarized. We reprint this portion of the report in full as follows:

The original contract provided that the contractor should paint the gates with two coats of red lead, at his expense, and with a third coat of some other pigment to be furnished by the Isthmian Canal Commission, applied to the gates at the contractor's cost. To provide for more complete protection, this agreement was modified and at Gatun arrangement was made for an additional coat, making two of red lead and two others, instead of one. These additional coats consisted, one of United States Navy anticorrosive and one of antifouling paint, and were applied to those parts of the gates in the lower locks which are constantly under water. On the remaining gates at Gatun two coats of equal parts of graphite and red lead were applied.

It was intended that no red lead should be used at Pedro Miguel except for the upper guard gates, the protection to consist of three coats of damp-proof paint. At Miraflores the gates in the lower lock from elevation —6 to the bottom were to be given two coats of red lead, followed by one coat of anticorrosive and one coat of antifouling paint. All the other gates were to be given three coats of a proprietary paint consisting of a hydrocarbon mixture.

On account of delay in receipt of the damp-proof paints intended for use on the Pedro Miguel gates, some of these were painted with hydrocarbon paint from Miraflores and, conversely, a few of the Miraflores gates were painted with damp-proof paint. In all of these cases the third and fourth coats were applied by the Commission forces.

Due to impurities in the water of Gatun Lake, none of the paints, except the bitumastic which was applied directly to the metal on small sections of several of the gates of Gatun, has proved satisfactory, and the paint on those parts which are constantly under water is in very poor condition.

An Unusual Type of Storm-Sewer Outlet at Passaic, N. J.

By R. F. Odell*

The unusual storm-sewer outlet here described was designed in the spring of 1914 by the writer.

In improving the street where these outlets are located, it was found necessary to make some permanent provision for caring for the storm water concentrated at this point by a 24-in. vitrified pipe sewer. This sewer had been designed several years earlier to carry the surface drainage of an area of about 40 acres of partially improved land with an average slope of 20 ft. per 1000. As there was no suitable place of discharge available at the time, an outlet into the street gutter was constructed at the point shown by Fig. 1.

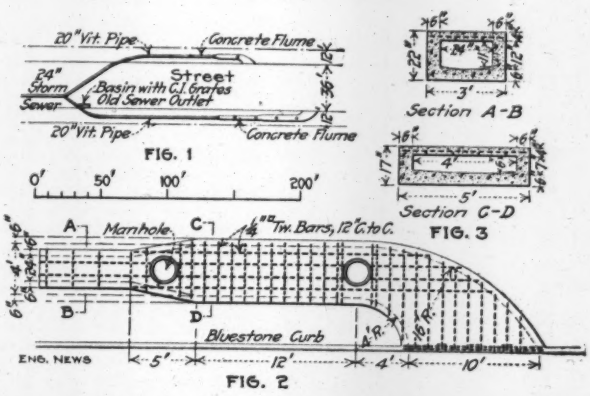


FIG. 2

This outlet was constructed in the form of a catch basin, into which the 24-in. sewer discharged. The point at which the pipe ended was about 5 ft. below the opening at the surface of the street. In times of heavy rains, the great volume of water rushed into the basin with such force as to lift the heavy cast-iron grates completely off the opening, tearing up the roadway and cobble gutters and causing considerable other damage.

When it was decided that the street be improved, including the paving of the road with asphalt-bound macadam, the advisability of constructing a relief sewer discharging into a nearby stream was discussed. It was finally decided that it would be necessary to adopt some other method, and the one here described was followed.

As shown in Fig. 1, two lines of 20-in. vitrified sewer pipe were laid from the existing manhole at the end of the 24-in. line to a point beneath the center line of the flagging on each side of the street. From these points, reinforced-concrete flumes were constructed leading to the gutters. In order to keep the grade of the flumes to the minimum allowable slope of 0.5 in 100, and to bring the two grades, of sewer and gutter, to a proper intersection, it was necessary to place the outlets at some distance from the old opening, as shown in Fig. 1.

The method of reinforcement and a plan of the flumes is shown by Fig. 2, and in Fig. 3 are shown sections at two designated points. A shallow section was necessary owing to the short distance between the run of the flume and the surface of the sidewalk. This was compensated by increasing the width as shown. It may be well to remark here that where shallow outlets or flumes of this type are constructed, it is quite necessary to place manholes or other suitable openings at close intervals to facilitate cleaning and inspection.

At the time the outlets were designed, the objection was raised that the velocity at the openings at the curb might be so great as to cause the water to rush into the street a considerable distance, and perhaps cause as much trouble as the old method. The outlets are, however, giving very satisfactory service, and have as yet shown no fault in either design or construction.

The writer personally knows of but one other flume or outlet of this type, but has heard of one or two others in this vicinity. The present city engineer, Colin R. Wise, was the first to use the idea in this section. The work was done under the writer's direction by the De Vogel Contracting Co.

*Assistant Engineer, City of Passaic, N. J.

A Novel Street-Cleaning Plan for New York City

A well conceived plan for removing snow from the streets of New York City in the winter of 1914-15 was presented to the Board of Estimate and Apportionment for approval under date of Nov. 10 by J. T. Fetherston, Commissioner of Street Cleaning.

Novel features of the plan are "snow fighting" to prevent an accumulation of snow; the largest possible use of the sewers of the city to convey the snow to the waters surrounding the city; and an emergency force of 15,000 men per shift of eight hours. Besides removal through the sewers, cart haulage by contract would be employed. In some sections, instead of removal the snow would be pushed from the center of the streets by motor-driven or horse-drawn plows. Apparently the plan also contemplates the use of the sewer to carry away snow removed by the street-railway companies from between their tracks.

An important factor would be the assignment to snow-removal work of men from other city departments, including extra clerks, inspectors, checkers, etc. The police would render aid in calling out the emergency force.

As an inducement to secure emergency men and efficient work, high rates of pay are proposed, with the choice between 30c. per hr. for regular work and 40c. per hr. for a set task accomplished within a time limit. Regular employees of the street-cleaning department working overtime would be paid regular wages for overtime, "and in addition, if the set tasks are accomplished within the time limits specified," 50% extra. To encourage men from other city departments to accept snow-work assignments, the plan provides for paying them 50% above their regular wages or salaries.

The snow schedule for 1914-15 covers a total length of 734 miles of streets and a total area of 25,830,743 sq.yd., in the boroughs of Manhattan, the Bronx and Brooklyn. Of these totals, it is proposed to use the "snow-fighting" force (to keep the snow from accumulating) on 542 miles, or 19,106,437 sq.yd.; the "snow-removal" or "contract schedule" on 94 miles, or 3,304,009 sq.yd.; and the "street-railway schedule" on 97 miles, or 3,420,297 sq.yd.

An analysis of the snowfall records for the years 1905-14, inclusive, shows the following:

	Maximum	Average
Number of storms per winter	7	4
Duration of single storm, hr.	33	13.5
Rate of fall per hour per storm, in.	0.9	0.5
Depth of snow per winter, in.	42	21.3

Conclusions Drawn from Official Test of the San Francisco Refuse Incinerator

A note to the effect that M. M. O'Shaughnessy, City Engineer of San Francisco, had recommended the rejection of the Islais Creek refuse incinerator after a test to determine whether it fulfilled the contract requirements, appeared on p. 1041 of our issue of Nov. 19. The conclusions on which this recommendation was based, as summarized by Mr. O'Shaughnessy in his official report, are printed below.

Although our note of Nov. 19 stated that the cost of incineration was in excess of the contract guarantee, the last paragraph of the quotation from Mr. O'Shaughnessy's report, given below, states that the cost guarantee "is fully met." It appears from the report that the actual cost of operating the incinerator was $1.33 per ton, as stated in our note of Nov. 19, but that after allowing for (1) steam produced, (2) excessive labor cost due to lack of refuse to operate the furnaces 22 out of 24 hours, and (3) the amount paid the city by scavengers for refuse delivered to the furnace, the contractor's guarantee of 25.5c. per ton of profit is considerably exceeded.

Mr. O'Shaughnessy's summary of conclusions drawn from the tests of the incinerator is as follows:

The plant does not meet the first guarantee that no nuisance is created in its normal operation.

The plant does not meet the second guarantee that no odors, obnoxious gases, smoke or dust escape from the building or the chimney.

The plant does not meet the third guarantee in the requirement that the temperature in the combustion chambers shall at no time fall below 1250° F. It does meet this guarantee in that the average temperature maintained in the combustion chambers is in excess of 1500° F.

The plant does not fully meet the fourth guarantee that the residue from the furnaces shall be thoroughly burned hard and free from organic matter.

The plant fully meets the fifth guarantee that the flues, dust collectors, furnaces and combustion chambers shall be so arranged that it shall not be necessary to shut down any one unit for more than 48 hr. in any one week in order to thoroughly remove dust and ashes.

The sixth guarantee that the pounds of refuse per hour which will be incinerated per square foot of grate area shall not be less than stated in the contractor's bid is fully met.

The seventh guarantee that the average cost per ton at each plant for incinerating refuse shall not be greater than stated in the contractor's bid is fully met.*

In his letter transmitting the report from which the foregoing conclusions are taken, Mr. O'Shaughnessy recommended the rejection of the incinerator, under a contract provision to the effect that if the plant did not fulfill all the contract guarantees the city might refuse to accept the plant, in which case the contractor should return to the city within 30 days all money paid to him by the city on account.

On Monday, Nov. 23, a protest against this recommendation was lodged with the Board of Public Works by the contractor, the Destructor Co., of New York City.

⁂

Ohio State Highway Contracts let on Oct. 30 amounted to nearly half a million dollars, covering the construction of 32.4 miles in 15 counties. This makes the total for the year over $7,000,000, and it is expected before the end of the year another quarter of a million dollars in contracts will be awarded. This does not include about $100,000 done by state on force-account work

*The contract guarantees were stated in detail on p. 116 of "Eng. News," of Jan. 15, 1914, at the end of an illustrated description of the incinerator.

NEWS NOTES

A Severe Freight Wreck at Harward's Station, Me., which officials of the Maine Central R.R. claim to be the worst in the history of the road, occurred on Nov. 17. Of the 50 cars in a westbound train, 31 were piled up when a car wheel broke.

A Steamship Collision in New York Bay occurred when the 26,000-ton outward-bound White Star steamship "Baltic" passed too closely on the port side of the 3000-ton "Comal," of the Mallory Line, bound for Galveston, Tex., on the afternoon of Nov. 18. The "Baltic" had drawn up abeam of the smaller vessel and was leaving her astern, when the "Comal" swerved in, her bow raking along the side of the "Baltic," due to the suction caused by the passage of the big steamer. Although she was not leaking, the bow plates and a deck beam of the "Comal" had buckled. The "Comal" returned to her pier and arrangements were made for sending her 60 passengers to Galveston by train. The "Baltic" was uninjured and continued her voyage.

Two Water Mains Were Broken in Greater New York, one in Varick St., Manhattan, that flooded the new Seventh Ave. subway excavation, and the other in Brooklyn, as the result of a heavy rainfall on Nov. 15. At Varick and Canal St. a 20-in. main which had been undermined broke, causing a 12-in. high-pressure pipe to give way. The foundation of a 10-ft. sewer on Canal St. was scoured, cracking the piping and permitting the water inside the sewer to issue into the excavation at the Canal St. end, to a depth of 20 ft. The water in that district was turned off as soon as possible and the high-pressure main repaired. Three hours after the break, the 20-in. main had been repaired. Cobblestones were then dumped into the excavation to form a foundation for the replaced sewer, and a temporary connection made to stop the flow into the excavation. The break in Brooklyn was in a 36-in. water main on Bushwick Ave. Sidewalks were ripped up and holes torn in the street, cellars were flooded, and the water stood at street intersections in some places 3 ft. deep. The rush of water from a sewer main through a temporary wood flume in the excavation of the Flatbush Ave. and Eastern Parkway subway, Brooklyn, tore out the sides of the flume, on the same night, and for more than three hours water poured into the excavation, which was filled to a depth of 8 ft.

The East Approach to the Municipal Bridge across the Mississippi River at St. Louis is at last to be built. On Nov. 3 a bond issue of $2,750,000 was authorized by vote of the citizens of St. Louis; this bond issue had been defeated at the three previous elections. For two years the rest of the bridge has been completed awaiting the east approach. The new construction will consist of a steel trestle, including a combined railway and highway approach 2363 ft. in length, a railway approach 10,975 ft., and a highway approach 2943 ft. in length. Plans are in preparation and work will probably start within the next 90 days. No contracts have been prepared to date.

Exhibition of Street Cleaning Machinery and Apparatus—A very interesting and instructive exhibit of apparatus for cleaning streets is being held at the Armory of the First Regiment of Field Artillery, 68th St. and Broadway, New York City, during the present week. About 40 manufacturers are exhibiting the most recent types of municipal street cleaning machinery, covering all kinds of apparatus from cans and hand sweepers to the largest automobile trucks and pressure street flushers. On Monday, Nov. 23, there was a street parade of exhibits and of the city's street cleaning forces. Each day of the exhibition there are band concerts, moving pictures and lectures. The exhibition is being held under the auspices of the New York City Department of Street Cleaning, of which John T. Fetherston is chief.

The Panama Canal Construction has required the employment of 45,107 contract laborers which have been brought to the Canal Zone from other countries by the Isthmian Canal Commission. Of these contract laborers, 11,873 came from European countries, 31,071 from the West Indies, and 2163 from Central America. The great influx of contract labor was in 1906-1907. In the first named year 12,600 laborers were brought into the zone, and in the following year 14,944. Since September, 1911, the Zone government has been engaged in returning to their homes the contract laborers; and as there is a dearth of employment on the isthmus at the present time, the Panama Canal has lately been offering free transportation to their home countries to practically all ex-employees

now out of work. The total number of European laborers employed by the Panama Canal and the Panama R.R. at present is about 1200.

The Cleveland Lake-Front Case has been decided in favor of the city and against the railway companies, after 21 years of litigation. The decision, which was rendered by the United States Supreme Court, on Nov. 16, gives the city possession of 30 acres of land claimed by the Cleveland & Pittsburgh, the Pennsylvania, the Cleveland, Cincinnati, Chicago & St. Louis, and the Lake Shore and Michigan Southern railways. The railways claimed title to the land under a 99-year lease made in 1849, granting the right to lay tracks on what is now West Ninth St., for which right the city was paid $100,000. Subsequently, the city filled in the lake to the north of West Ninth St. The railways claimed the filled land, but the United States Supreme Court holds that the city has title to all improvements made since 1897. The city now has possession of the lake front from West Ninth St. to the mouth of the Cuyahoga River, with the exception of that part lying between West Ninth St. and West Third St.

Bridge Reconstruction, Kansas City—The Mercantile Club, of Kansas City, Kan., held a meeting on Nov. 10, to determine the consensus of opinions of different bodies and associations who are interested in the reconstruction of the Central Ave. bridge across the Kaw River. This bridge is a double-deck structure carrying the Metropolitan cars overhead and a traffic way underneath. Owing to the fact that the bridge does not comply with the requirements of the drainage Board as to length, height above levees, etc., it has been ordered reconstructed by Judge Smith. The Metropolitan engineers have made an estimate of $115,000 on cantilever type, using the present two piers by extending them down below possible scour line. County Engineer L. R. McAlpin has made an estimate of $300,000 on the same type of bridge as is there at present. This would require two new piers and three new spans. The real-estate men and residents adjacent to Central Ave. have formed an association and are demanding a high line bridge which, it is estimated, will cost $450,000. No action has as yet been taken in this mater.

The First Brick Road in Montgomery County, Ohio, was recently completed, as a part of a projected outlay of $180,000 to be expended on similar roads radiating from Dayton, the county seat. The road is one of four radiating main market and intercounty highways. The first, the Covington road improvement, is little more than a mile in length. Two and a half miles of paving is under way on the Dayton-Cincinnati route, a trifle less on the Dayton-Troy road (a part of the through route to Toledo), and one mile on the Dayton-Greenville road. An additional mile and a half of the last named road will be graded this year in preparation for a brick surface next year. This year's program entails the expenditure of about $180,000 owing to difficult grades which have been reduced and the generally substantial scale of the undertaking. Of this, the state is furnishing one-half, a recognition of the extent of the through traffic which is destined to use these main market intercounty roads. The remaining 50% of cost is divided so that 25% falls on the county, 15% on the various townships and 10% upon the owners of adjacent land. The work has been done under the joint direction of Division Engineer Harwood Lersch, of the State Highway Department, and Victor C. Smith, surveyor of Montgomery County.

Opposition of Organized Labor to efficiency systems of shop management was voiced at the Convention of the American Federation of Labor in Philadelphia, Nov. 16. The convention adopted a resolution urging the Executive Council to continue its efforts to have abolished so called efficiency systems by which workmen are speeded up. Another resolution adopted urged that effort be made to have the federal eight-hour law made to apply to the manufacture of all materials used in government work. The ruling of the Attorney-General has been that the eight-hour law does not apply to the manufacture of materials which the government may purchase for its own use. The Federation will attempt to secure a contrary decision from Attorney-General Gregory, and if this is unavailing an effort will be made to amend the law. The Federation further went on record in regard to workmen's compensation laws with the demand that at least 66⅔% of wages should be the rate paid to relatives of workmen killed during their employment. The compensation should be paid to the widow during her widowhood and to children under eighteen years of age. The convention further favored the prohibition of employer's liability insurance companies and also permission to employers to carry their own insurance, thus making state insurance compulsory. To prevent discrimination against married men by employers engaging help, the convention voted to recommend that where a workman who is killed has no wife or other dependent, the compensation should be paid to the state.

Railway Valuation in the United States will cost about $12,500,000 to complete, according to a statement by Judge Chas. A. Prouty, Director of Physical Valuation, at the annual convention of the National Association of Railway Commissioners on Nov. 19. The valuation of the telegraph lines will cost $1,000,000 to $1,250,000 more. Regarding the probable use of the valuation when made in fixing rates and its possible use as the basis of government purchase, Judge Prouty said:

This valuation is national, but is to be made along State lines. When completed, it undoubtedly will supersede all state valuations that have been made. It will be the basis for the making not only of interstate, but of state freight rates.

Probably the most important practical question before this country today is its treatment of our railroads. Is the Government to take over these agencies of transportation, or shall they be left to the operation of private capital? Assuming that we are to leave the discharge of this function to private enterprise, what rates shall be accorded in order that the present investment may be fairly dealt with and that such further investment will be induced as will render possible the proper development of these facilities?

In the past this question has not been acute, for rates have been sufficient to maintain railroad credit, but it is evident that the time is close at hand, if it is not already here, when there is grave doubt as to whether these rates must not be generally increased. This question cannot be satisfactorily answered until there is a national valuation of these properties.

Consider the enormous amounts involved. An error of 6 or 7% in the valuation means nearly or quite a billion dollars. The interest at 6% on that sum is $60,000,000. Whether this valuation is to be used as the basis of rates to be allowed these carriers and paid by the body of our people, or whether, perchance, it may become the measure of the price to be paid by the Government for this property, that error would mean either to the railroads or to the people every year three or four times the cost of the entire valuation.

PERSONALS

Mr. Floyd F. Woods has been appointed Sales Manager of the Epping-Carpenter Pump Co., Pittsburgh, Penn., succeeding Mr. R. Bowen, resigned.

Mr. George W. Hunt, formerly President and General Manager of the Canadian Asphalt Co., Ltd., has been appointed Sales Manager for John Baker, Jr., dealer in asphalt, bituminous products and road oils, 10 South La Salle St., Chicago, Ill.

Mr. Thomas E. Mitten has been elected President of the Philadelphia Rapid Transit Co., succeeding the late Charles O. Kruger. Mr. Mitten has been Chairman of the Executive Committee since 1911. He was formerly President of the Chicago City Ry.

General Hartwig von Beseler, the German commander who besieged and captured Antwerp, has been awarded the degree of Doctor of Engineering by unanimous vote of the faculty of mechanical engineering of the Technical High School of Hannover.

Mr. H. C. Hequembourg has resigned as General Purchasing Agent of the American Locomotive Co., New York City. Until further notice the purchasing and storekeeping departments will be under the jurisdiction of Mr. Leigh Best, Vice-President.

Mr. Edward M. Bigelow, M. Am. Soc. C. E., State Highway Commissioner of Pennsylvania, who was recently indicted by the authorities of several Schuylkill County townships for alleged criminal negligence in not caring for certain roads, has won his case and the indictment has been quashed. The court held that evidence showed the State Highway Commissioner had done his best in apportioning state funds to the townships, and that his action was in no way criminal.

Col. H. G. Prout, M. Am. Soc. C. E., former President of the Union Switch & Signal Co., New York City, has been elected President of the Hall Switch & Signal Co., with headquarters in New York City. Mr. Wm. P. Hall is now Vice-President and Chairman of the Executive Committee. Col. Prout was associated with the Union Switch & Signal Co. as Vice-President and General Manager for several years, and was elected President to succeed the late George Westinghouse.

Mr. Charles W. Mullen, a graduate in civil engineering of the University of Maine, class of 1883, and for several years actively engaged in engineering construction work, has been nominated a member of the Maine State Public Utilities Commission, in place of Samuel W. Gould, of Skowhegan, whose nomination the legislature refused to confirm. The Bangor "Daily News" states that the opposition to Mr. Gould was based largely upon the widespread belief that the board should not be composed entirely of lawyers. Mr. Mullen has been twice Mayor of Bangor, and is a prominent business man of that city.

Mr. R. S. Buck, M. Am. Soc. C. E., has resigned as Chief Engineer of the Dominion Bridge Co., Montreal, Que., to become Engineer of Maintenance-of-Way of the New York Rys., succeeding Mr. William T. Dougan, M. Am. Soc. C. E., whose resignation was noted in our issue of Nov. 5. Mr. Buck was associated with Sanderson & Porter, Engineers and Contractors, New York City, until February, 1913, when he went to Montreal. From 1899 to 1902 he was Chief Engineer of the Manhattan and Queensborough Bridges, New York City, and from 1904 to 1907 was Consulting Engineer in charge of these bridges.

Mr. Delos F. Wilcox has recently been appointed Deputy Commissioner of the Department of Water Supply, Gas and Electricity, New York City. Mr. Wilcox was formerly Chief of the Bureau of Franchises of the Public Service Commission for the First District, New York, but since July 1, 1913, has been engaged in private practice as a consulting franchise and public-utility expert. Last summer he was employed to take charge of the valuation of the private water companies operating in New York City, such companies being subject to regulation as to rates and services by the Commissioner of the Water Department. In addition to the regular duties of his new office, Mr. Wilcox will carry forward to completion this valuation work.

Mr. J. G. Sing, M. Can. Soc. C. E., has resigned as District Engineer, Canadian Public Works Department, Toronto, Ont., to engage in private practice as a Consulting Engineer. He will also continue to be Consulting Engineer of the Toronto Harbor Commission. Mr. J. M. Wilson, Assoc. M. Can. Soc. C. E., Assistant Engineer, is Acting District Engineer. It is reported that Mr. C. H. Mackenwson, Assoc. M. Can. Soc. C. E., present Resident Engineer on the Canadian Government work for the improvement of Toronto harbor, will temporarily assume charge of the Government's share in the work, which involves an outlay of $7,000,000. Mr. Sing has been District Engineer for the past ten years and as such has had charge of harbor work in eastern and northern Ontario, including practically all the waterfront on the Great Lakes.

OBITUARY

C. W. Cooper, a levee contractor of Lake Providence, La., was crushed to death in an automobile accident at Waterproof, La., Nov. 6.

L. Freeman Day, former Vice-President and Manager of the Minneapolis & St. Louis R.R. and the Iowa Central Ry., died Nov. 15. He was 56 years old.

W. W. Cain, a bridge contractor for the Chicago, Rock Island & Pacific Ry., was struck and killed by a train at Vona, Colo., Nov. 2. He was a resident of Denver, Colo., and widely known as a mine owner.

James L. Brusse, a young civil engineer of Vancouver, B. C., was killed recently on a hunting trip in British Columbia. He was 28 years old, a native of Denver, Colo., and a civil engineering graduate of the University, of Denver.

Brig.-Gen. William A. Jones, Corps of Engineers, U. S. A., retired, died Nov. 10 in Washington, D. C. He was born in Missouri in 1841 and graduated from the United States Military Academy at West Point in 1864. His engineering experience included service as Assistant Professor at West Point, chief of an exploration party in Northwestern Wyoming and the Yellowstone National Park, engineer in charge of the construction of various lighthouses. He was retired in 1906, having reached the rank of colonel.

Lucian A. Taylor, M. Am. Soc. C. E., Consulting Engineer, Worcester, Mass., died Nov. 19, at his home in Worcester, Mass. He was born in Harrisville, R. I., 68 years ago. For 17 years he was an employee of the city engineer's office in Worcester, and for three years was Water Commissioner of the city. Following his work in Worcester he was for some time a consulting civil engineer and contractor, of Boston, Mass. He is survived by a widow, a son and a daughter, who is the wife of Prof. Chas. M. Allen, M. Am. Soc. C. E., of the Worcester Polytechnic Institute.

James Geddes, Assistant to the General Manager of the Louisville & Nashville R.R., of Nashville, Tenn., died Nov. 13. He was born Jan. 16, 1828, at Portsoy, Scotland. From 1846 to 1850 he was with the civil engineering department of the Great Western Ry. in England. On coming to America he was for eight years, 1851 to 1858, a location and construction engineer of the Louisville & Nashville R.R. He then entered the operating department and for many years was superintendent of the Nashville division, and since 1901 has been Assistant to the General Manager.

ENGINEERING SOCIETIES

COMING MEETINGS

AMERICAN SOCIETY OF REFRIGERATING ENGINEERS.
Nov. 30-Dec. 1. Annual meeting at New York. Secy., W. H. Ross, 154 Nassau St., New York City.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Nov. 30-Dec. 5. Annual meeting at Jacksonville, Fla. Secy., Prof. Selskar M. Gunn, Massachusetts Institute of Technology, Boston, Mass.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
Dec. 1-4. Annual meeting at New York City. Secy., Calvin W. Rice, 29 W. 39th St., New York City.

AMERICAN INSTITUTE OF ARCHITECTS.
Dec. 2-4. Convention at Washington, D. C. Secy., D. Knickerbacker Boyd, The Octagon, Washington, D. C.

ASSOCIATION OF AMERICAN PORTLAND CEMENT MANUFACTURERS.
Dec. 7-10. Annual meeting in New York City. Secy., Percy H. Wilson, Bellevue Court Bldg., Philadelphia, Penn.

AMERICAN MINING CONGRESS.
Dec. 7-11. Annual meeting at Phœnix, Ariz. Secy., J. F. Callbreath, Denver, Colo.

NATIONAL SOCIETY FOR THE PROMOTION OF INDUSTRIAL EDUCATION.
Dec. 9-12. Convention at Richmond, Va. Secy., C. A Prosser, 140 W. 42d St., New York City.

SOCIETY OF NAVAL ARCHITECTS AND MARINE ENGINEERS.
Dec. 10-11. Annual meeting at New York City. Secy., D H. Cox, 29 W. 39th St., New York City.

AMERICAN ROAD BUILDERS' ASSOCIATION.
Dec. 14-18. Convention at Chicago, Ill. Secy., E. L. Powers, 150 Nassau St., New York City.

AMERICAN SOCIETY OF AGRICULTURAL ENGINEERS.
Dec. 28-30. Annual meeting at Chicago, Ill. Secy., F. M. White, Madison, Wis.

American Iron and Steel Institute—At the annual convention, recently held in Birmingham, Ala., it was decided to hold the 1915 meeting at Youngstown, Ohio.

Washington (D. C.) Society of Engineers—The annual dinner of the Society was held on Nov. 19. The attendance was about 250. The Secretary is John C. Hoyt, 1330 F St. Northwest.

New York Section of American Water Works Association—The third meeting will be held at the Manhattan Hotel, New York City, on Dec. 3, at 12:30 p.m., when lunch will be served.

Northwestern Road Congress—The officers elected at the annual meeting at Milwaukee, Wis., were: President, George W. Cooley, State Highway Engineer, St. Paul, Minn.; Vice-Presidents, John Hazlewood, Madison, Wis., P. C. McArdle, Springfield, Ill., and Otto Albertus, Doland, S. D.; Secretary-Treasurer, James P. Keenan, Milwaukee, Wis. A report of the meeting was given in our issue of Nov. 5.

Cleveland Chapter of the American Institute of Architects—At the November meeting of the Chapter, in the Hotel Statler, Cleveland, it was resolved to recommend to the 81st General Assembly of Ohio the passage of a law recommending the practice of architecture, with the purpose of placing the profession upon the same plane as the professions of law and medicine; in general, the law to admit men to the practice of architecture by examination and to confer upon them the right to use the title of architect.

Illinois Water Supply Association—A special meeting was held in Chicago on Nov. 11 to take steps for affiliating with the American Water Works Association, as the Illinois section of the latter. This was in accordance with a ballot vote of the association, with a large majority in favor of such affiliation. An excursion was made to the city waterworks pumping stations at Chicago Ave. and at Montrose Ave., and then to the new water-filtration plant at Evanston, Ill. At the business session, a paper on this plant was read by Langdon Pearse, and a talk on "Water Purification and Sewage-Disposal Plants in Europe" was given by Edward Bartow, the Secretary.

City Managers' Convention—The program for the First Annual Convention of City Managers of the United States, to be held at Springfield, Ohio, Dec. 2-4, has been issued, and includes the following addresses: "The Training of the City Manager," Henry M. Waite, City Manager, Dayton, Ohio: "Coöperation between the Departments of the City Government," William F. Robertson, City Manager, Sumter, S. C.; "The Coöperation of the Citizen, and How to Obtain It," Ossian A. Carr, City Manager, Cadillac, Mich.; "Who Benefits from City Government, and How?", Kenyon Riddle, City Manager, Abilene, Kansas; "The Handling of Public Service Corporations and Contractors," Claude E. Campbell, City Manager, Big Rapids, Mich. The Convention headquarters will be at the Springfield Commercial Club, Fairbanks Building.